STEEL H PILES

BETHLEHEM STEEL

BETHLEHEM STEEL

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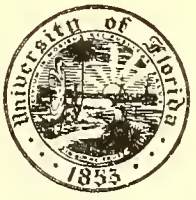

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STEEL H PILES

BETHLEHEM STEEL CORPORATION, Bethlehem, Pa.

Section

1

HISTORY OF STEEL H-PILES

The use of steel bearing piles made from structural shapes started about 1899, although the use of metal piles, both cast-iron and wrought-iron, dates back over 85 years.

The earliest steel piles were in the form of I-beams and fabricated sections made from plates and angles. These steel bearing piles were used first in the Middle Western States in what proved to be a successful effort to meet the serious problem caused by the undermining of bridge piers and abutments on pile foundations in compact gravel and sand. These soil conditions limited the depth to which piles other than steel could be driven. It was demonstrated that steel piles could withstand extremely hard driving and could penetrate to depths safely below the scouring action of streams. The steel piles also solved another serious problem by providing the necessary resistance to damage from ice floes.

A typical instance of such a condition was the difficulty in providing safe and substantial bridge crossings along the Platte River, in Nebraska. This river is wide but shallow and has an extensive drainage area. Because of these characteristics, early spring thaws cause ice floes to develop out of all proportion to the discharge capacity of the stream. At such times, flooding of the river opens new channels by a violent scouring action, which in many cases undermines the bridge structures. The only salvation for such structures was the development of foundation piles capable of being driven well below the possible scour level. In this instance, I-beams and fabricated steel piles provided the only solution.

After 1908, when Bethlehem Steel rolled the first steel H-column section in this country, the H-section rapidly became recognized as ideally suited for use as piling.

The early use of steel piles, primarily as a means of combatting stream scour and resisting attacks from floating ice, also demonstrated that greater loads could safely be carried by the deep-driven steel piles than by other types of piles. Tests proved that with sufficient penetration in deep beds of sand or gravel, the bearing capacity of a steel H-pile could be taken to be its strength as a column, with a length measured from the region of estimated maximum scour to the top of the pile. The high bearing value of the steel H-pile permitted its use with initial economy in addition to providing long-term economy by affording ample security against scour and ice.

The originally used I-beams and fabricated sections driven deep into the ground provided sturdy foundations, but the Bethlehem wide-flange H-sections solved the problem even more satisfactorily.

The success of steel H-piles was immediate, and the use of the older types of steel piles was soon abandoned. By 1932, several thousand bridges stood on steel H-pile foundations in the territory between the Continental Divide and the Mississippi River. In this area, there is complete diversity of soil and climatic conditions. Within recent years, there has been a great increase in the use of steel H-piles in other

parts of the country, which can be attributed in large measure to the excellent service rendered by these early installations. The use of steel H-piles has extended to practically every class of work for which bearing piles are required, and particularly where high bearing values can be developed. It is noteworthy that for the past 25 years, many monumental bridges have been constructed on H-piles at sites where there are extreme depths of very soft soil. In some cases, pile lengths have exceeded 200 ft, penetrating soft soil to reach rock, or other hard bearing stratum. Steel H-piles, today, are used for an extremely wide variety of structures, including bridges, buildings, trestles, viaducts, oil tanks, water tanks, grain elevators, mill buildings, industrial buildings and factories, steamship piers, marginal wharves, berthing facilities for naval vessels and floating drydocks, graving docks, shipways, transmission towers, anchor piles for bulkheads and dock walls, soldier beams for temporary excavations, and numerous other structures.

Bethlehem H-piles are rolled from steel conforming to American Society for Testing and Materials Specifications A7 and A36, and are also furnished in Bethlehem V Steels V45, V50, V55 and V60.

Each Bethlehem H-pile is a specially designed wide-flange section with flanges and web of equal thickness. Other Bethlehem wide-flange sections may be selected for use as H-piles, if desired. Bethlehem H-piles are available with a minimum copper content of 0.20 per cent, at a slight extra cost, should copper-bearing steel be desired.

A large, modern steam-air hammer rigged to drive steel H-piles.

Ten thousand tons of Bethlehem H-piles were used for the bridge piers and foundations of the Taunton River Bridge, Somerset, Mass.

Steel H-piles can be driven close together to sustain high loads.

Section 2

ADVANTAGES OF STEEL H-PILES

Strength

Steel is the only material used in piling that has a crushing strength comparable to that of hard rock. This allows the designer to develop the full bearing strength of the stratum once the pile is driven to refusal. This advantage is further emphasized by the performance of steel piles in carrying highly concentrated loads. H-pile foundations have been designed to carry loads as high as 225 tons per pile without difficulty. This strength makes possible the building of structures in locations impossible with other types of piling; the strength of steel H-piles permits them to be installed in greater lengths than is feasible with other materials.

Toughness

Driving through sand to hardpan, or into soft rock, and other soils, to develop high loads per pile in friction is readily accomplished; tough steel piles permit driving through unconsolidated soils containing timbers and other debris with a minimum of difficulty, even where long lengths of piles are involved. The high elasticity and bending strength of steel H-piles make them desirable for use in deep-water piers and other structures where varying lateral loads from wind, direct impact shocks, and other disturbing forces are likely to occur.

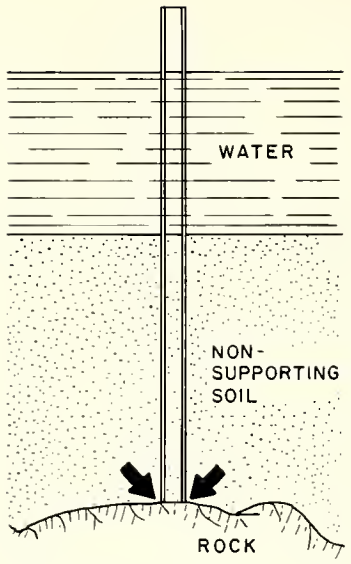
Economy

Available in a wide variety of sizes, each selected according to the expected load, H-piles permit economies not available in other types of piling. They are readily spliced in the field; welds develop 100 per cent of the compressive and bending strength of the pile. Jobsite fabrication costs are usually negligible, because the steel H-pile is complete in its as-rolled state, and is available in extremely long lengths. A steel H-pile in excess of 200 ft can be picked up and driven without imparting excessive handling stresses. And, as little soil is displaced when steel H-piles are driven, there is a minimum of disturbance to the foundations of nearby existing structures.

Long life

Steel H-piles usually will long outlive the economic life of the structures they support. They need no special protection from marine borers, termites, and dry rot, and do not spall and chip out.

Properly protected, steel piling will give many years of trouble-free service in waterfront locations.



Section 3

END-BEARING PILES

H-piles driven to hard rock

Steel is the only material used for foundation piles having an ultimate strength in compression comparable to that of hard rock. That is why H-piles are predominantly suitable for a wide range of foundation structures. An example would be bridge piers founded on rock lying at great depth below water level. H-piles have been used for such applications in lengths exceeding 200 ft, and carrying unusually high loads per pile. The piles can be driven on close centers, taking advantage of the high bearing capacity of the rock to a degree not possible with other types of piles.

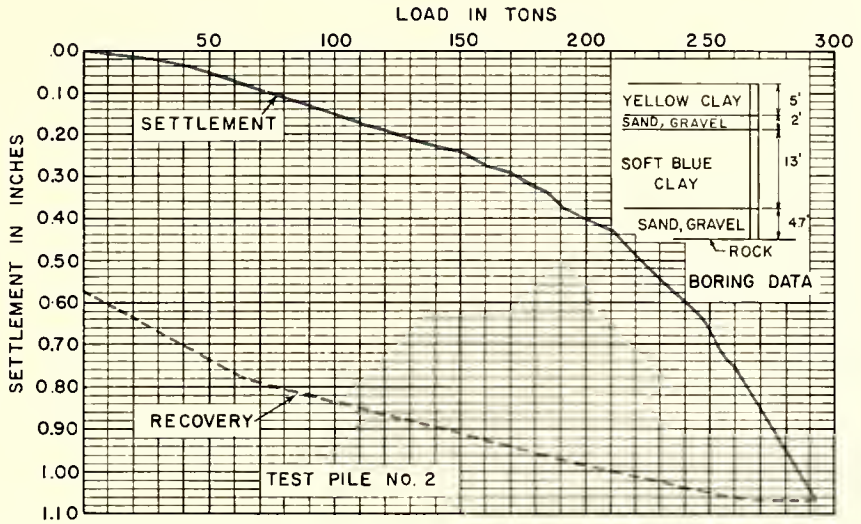
In many northern cities, bedrock is overlain by hardpan. This is true in such cities as New York, Boston, Buffalo, Detroit, and Chicago. In these areas many engineers require that the piles be driven through the hardpan to bearing on the rock. In most cases this is not necessary, since the hardpan, if homogeneous, will develop more than adequate bearing capacity. However, when penetration of the hardpan is specified, the piles must withstand extremely hard driving. Steel H-piles are well suited to this severe usage.

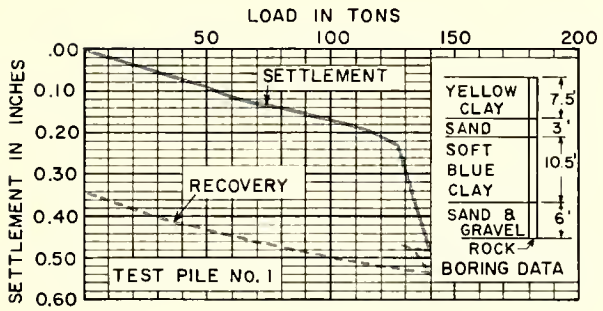
Some engineers are concerned about what may happen to the bottom end of an H-pile driven to refusal on rock. In 1935, we answered this question to our satisfaction by comparative tests at our Lackawanna plant—when we were designing the foundations for a new mill—using H-piles driven to rock. The results of the tests made at that time are shown in Figure 3A. They include borings, driving logs, load-settlement curves for two piles, and other pertinent data.

Note that Pile No. 1 was driven to rock to a final resistance of 11 blows per inch. When the pile was load-tested, it failed at approximately 140 tons. When it was pulled, the bottom end was found to be in perfect condition.

Bethlehem H-piles have been used as end-bearing piles in lengths up to 200 ft.

FIGURE 3A—H-PILE TESTS FOR STRIP MILL FOUNDATIONS, LACKAWANNA, N. Y.





Piles were Bethlehem BP 10 x 57 lb.

Piles were driven with a No. 1 Vulcan Steam Hammer.

Pile 1 was driven to practical refusal on the rock. No attempt was made to penetrate the rock.

Pile 2 was overdriven in an attempt to obtain absolute refusal in the rock. Examination of the pile after it was pulled showed that the bottom was distorted and twisted from this severe driving.

Loads were applied by a hydraulic jack acting against a steel frame loaded with steel ingots. The frame received additional support from four anchor piles. See page 53, Figure 14B, for a photograph of this test.

Settlement readings were made by means of an Ames gage.

Boring for Pile 1 was made about 100 feet away from the pile.

Boring for Pile 2 was made about 30 feet away from the pile.

DRIVING DATA

Pile No. 1	
Penetration	*No. of Blows*
0 ft 0 in. to 16 ft 0 in.	207
16 ft 0 in. to 19 ft 0 in.	17
19 ft 0 in. to 20 ft 0 in.	9
20 ft 0 in. to 21 ft 0 in.	8
21 ft 0 in. to 22 ft 0 in.	10
22 ft 0 in. to 23 ft 0 in.	10
23 ft 0 in. to 23 ft 5 in.	5
23 ft 5 in. to 23 ft 6 in.	10
23 ft 6 in. to 23 ft 7 in.	12
23 ft 7 in. to 23 ft 8 in.	11
23 ft 8 in. to 23 ft 9 in.	6
23 ft 9 in. to 23 ft 10 in.	6
23 ft 10 in. to 23 ft 11 in.	11

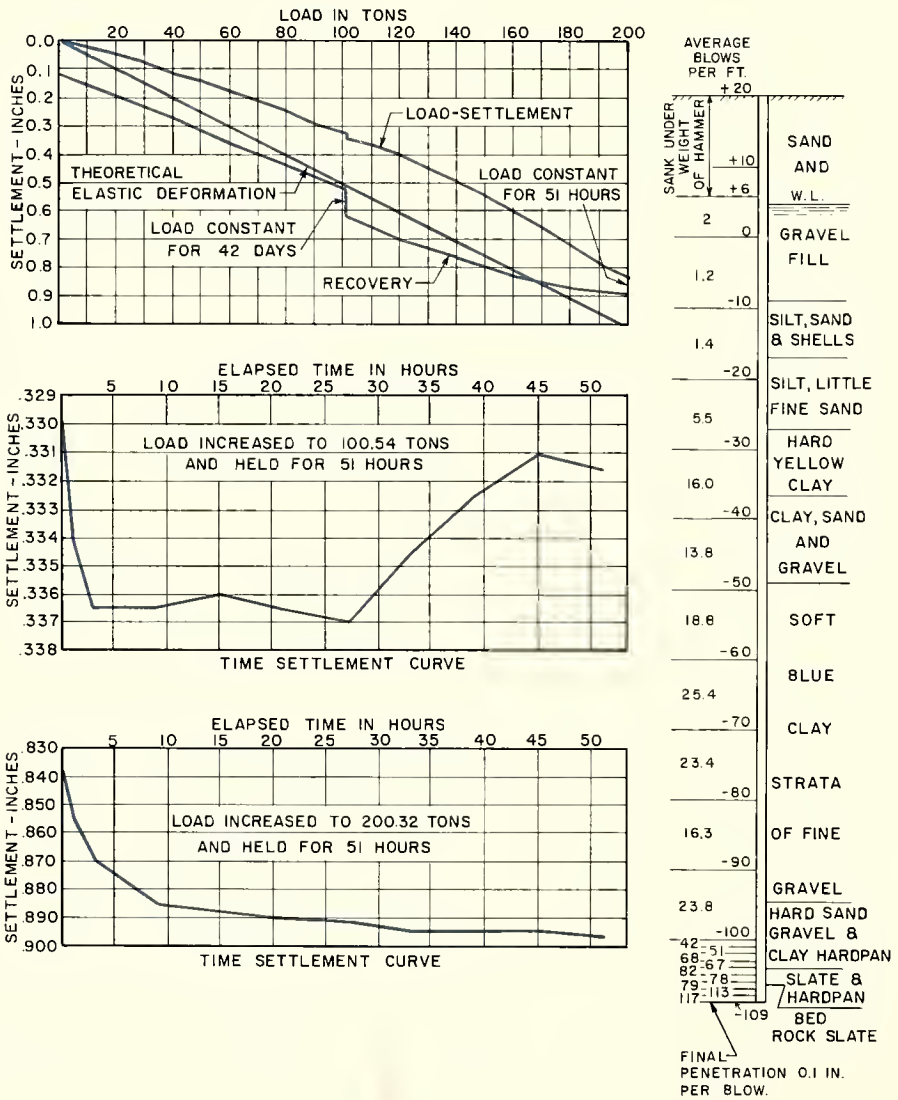
Pile No. 2	
Penetration	*No. of Blows*
0 ft 0 in. to 24 ft 0 in.	523
24 ft 0 in. to 25 ft 0 in.	214
25 ft 0 in. to 26 ft 0 in.	346
26 ft 0 in. to 26 ft 1 in.	30
26 ft 1 in. to 26 ft 2 in.	37
26 ft 2 in. to 26 ft 3 in.	42
26 ft 3 in. to 26 ft 4 in.	42
26 ft 4 in. to 26 ft 5 in.	38
26 ft 5 in. to 26 ft 6 in.	49
26 ft 6 in. to 26 ft 7 in.	49
26 ft 7 in. to 26 ft 8 in.	49
26 ft 8 in. to 26 ft 9 in.	43
26 ft 9 in. to 26 ft 10 in.	70

On the other hand, Pile No. 2 was deliberately overdriven, with a final resistance of 214 blows for 12 inches, then 346 blows for 12 inches, and finally 449 blows for 10 inches. The last inch required 70 blows. The load-settlement curve indicates failure at 293 tons, more than double the load for Pile No. 1. When the pile was pulled, the bottom end was distorted and twisted due to the severe overdriving. The significant fact is that overdriving and distortion of the bottom of the pile did not impair the load-carrying capacity of the pile. The rock was hard limestone, which the pile could not penetrate. Despite the fact that the top of the pile was driven down for approximately three feet after the bottom of the pile had reached rock it could not penetrate, the load-carrying capacity of the pile actually was doubled.

Since this test we have driven many thousands of H-piles at our Lackawanna plant, all of them to rock, and all of them designed for heavy loads.

FIGURE 3B—H-PILE TESTS FOR FOUNDATIONS OF 30 STORY OFFICE BUILDING

John Hancock Mutual Life Insurance Co., Boston, Mass.

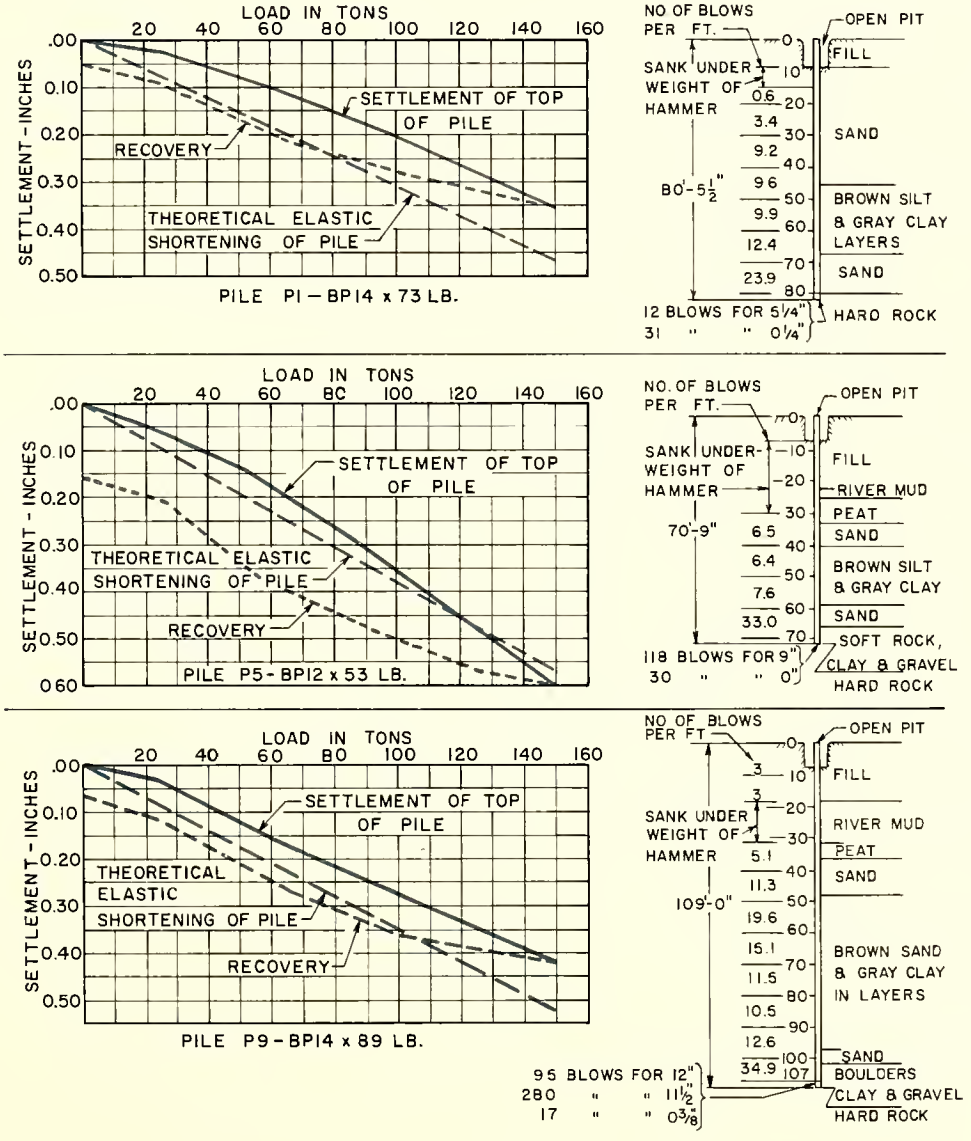

LOAD IN TONS
SETTLEMENT - INCHES
LOAD-SETTLEMENT
THEORETICAL ELASTIC DEFORMATION
LOAD CONSTANT FOR 51 HOURS
LOAD CONSTANT FOR 42 DAYS
RECOVERY
ELAPSED TIME IN HOURS
LOAD INCREASED TO 100.54 TONS AND HELD FOR 51 HOURS
TIME SETTLEMENT CURVE
LOAD INCREASED TO 200.32 TONS AND HELD FOR 51 HOURS
AVERAGE BLOWS PER FT.
SANK UNDER WEIGHT OF HAMMER
SAND AND GRAVEL FILL
W.L.
SILT, SAND & SHELLS
SILT, LITTLE FINE SAND
HARD YELLOW CLAY
CLAY, SAND AND GRAVEL
SOFT BLUE CLAY
STRATA OF FINE GRAVEL
HARD SAND GRAVEL & CLAY HARDPAN
SLATE & HARDPAN
BED ROCK SLATE
FINAL PENETRATION 0.1 IN. PER BLOW.

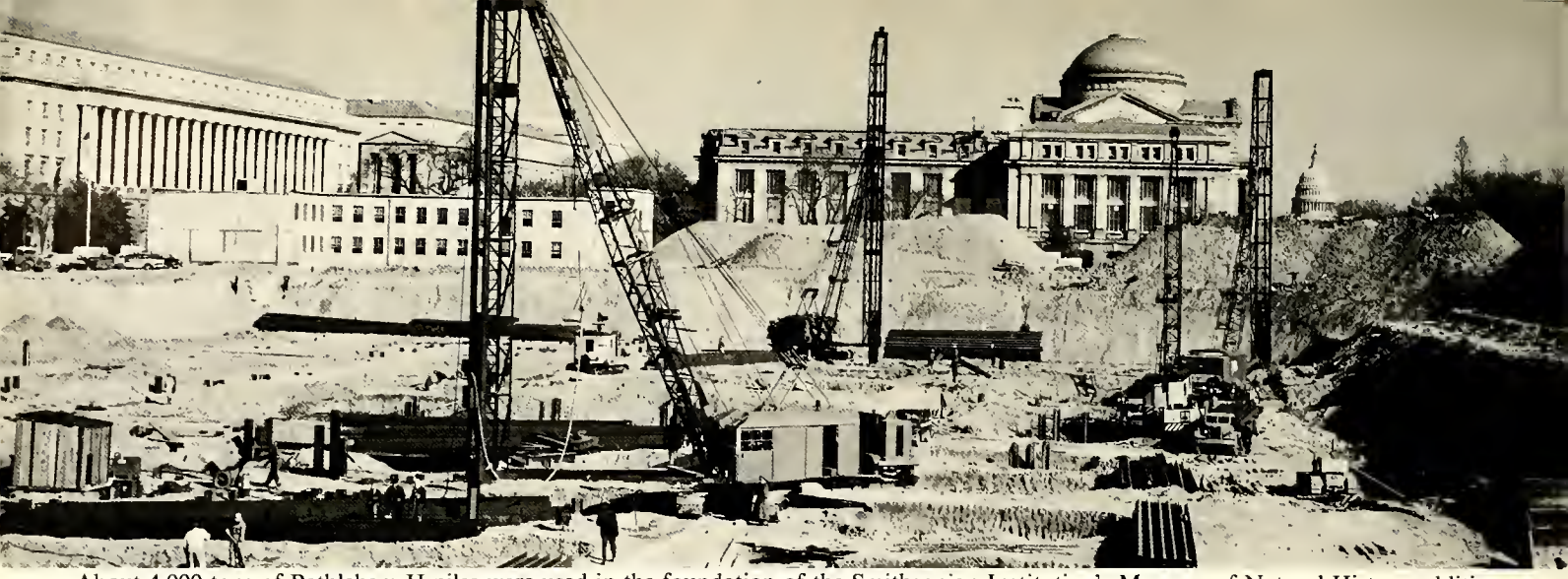
We do not recommend that piles be overdriven as described. The New York Building Code currently requires, as a guide for piles driven into rock, five blows of the hammer for the final quarter-inch of penetration. It is our experience that 20 blows with an adequate hammer for the final inch of penetration should develop full capacity of the pile, without undue distortion of the bottom end.

As a general rule it is not recommended that the bottom ends of H-piles be reinforced with points. They are not normally necessary, and add to the cost. Points are discussed in more detail on page 42.

When H-piles are driven to rock, test loads to failure invariably indicate that the piles do not fail until the yield point of the steel is equalled or exceeded. Pile No. 2 in Figure 3A was stressed to 35,000 psi. Tests by the Armour Research Foundation in Chicago demonstrated that a 14-in. x 117-lb pile failed at a stress of 37,200 psi; a 12-in. x 53-lb pile at a stress of 38,500 psi, and a 10-in. x 42-lb pile at

FIGURE 3C—H-PILE TESTS MADE BY NEW YORK HOUSING AUTHORITY





About 4,000 tons of Bethlehem H-piles were used in the foundation of the Smithsonian Institution's Museum of Natural History addition.

38,800 psi. Tests in Dearborn, Michigan, in 1936, resulted in failure of four 12-in. x 53-lb piles at stresses of 34,500 psi, 36,900 psi, 38,000 psi, and 38,600 psi.

Figure 3B shows the complete record of a test pile in Boston, including soil boring, driving log, load-settlement curve, and the time-settlement curve. The pile was loaded to 200 tons. The net settlement after removal of the test load was only 0.12 inch, and the stress in the pile was only 18,700 psi. This was less than the failure load.

Figure 3C shows similar data for three test piles in New York. Here the piles were loaded to only 150 tons, and the net settlements of 0.06 inch, 0.16 inch, and 0.07 inch, indicate that the piles were not loaded to failure.

H-piles in shale, hardpan, marl, and various soft rocks

When piles are driven to shale, hardpan, marl, or one of the soft rocks, the crushing strength of the bearing stratum must be considered. A pile of bulky cross section will not penetrate the surface of such material, and cannot be driven into it. The intensity of load on the small area will result in crushing of the surface and settlement of the pile. The bearing capacity of materials of this classification is very low as compared to hard rock. For instance, the New York Building Code specifies a presumptive bearing capacity for soft rock of eight tons per square foot, and of 12 tons for hardpan.

H-piles are especially suitable for use under these conditions, because they can be driven into the bearing stratum and develop not only point bearing but skin friction. This results in a greatly enlarged bulb of pressure with greatly increased bearing capacity. The project shown at the top of this page is an example of this condition. The consulting engineer had specified that the H-piles were to be driven a minimum of five feet into the underlying shale. This was done successfully, and load tests verified that the piles carried the specified load with an ample factor of safety.

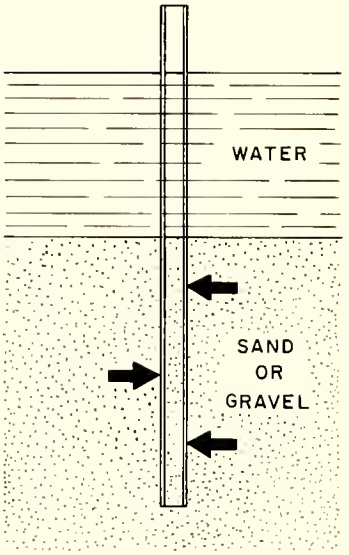
Load tests are of more than usual significance in these types of soil conditions, because occasionally the piles must be driven to greater resistance than a pile-driving formula indicates; the required amount of penetration to provide the necessary combination of point resistance and skin friction will not otherwise be obtained.

Civic Center Building, San Diego, California. Over 1,200 tons of Bethlehem H-piles were used as friction-piles in the foundation.

Section

4

FRICTION PILES



While it is well recognized that H-piles will develop extremely high loads per pile when driven to end bearing on rock, there still remain some misconceptions that H-piles are not suitable for use as friction piles. The reason for this undoubtedly is due to experience in certain cases where H-piles were inadvisedly driven into soils of poor consistency, such as soft clay, or loose organic silt.

Page 13 and succeeding pages show examples of friction piles that developed adequate resistance in soils classified as sand, sand-clay, silt-and-sand, sand-and-gravel, or fine sand.

H-piles in sand, gravel, or sand-and-gravel

The distinctive properties of sand, gravel, and sand-and-gravel, aside from size of individual particles, are permeability, incompressibility (unless in a loose, uncompacted state), high coefficient of friction, and low cohesive strength.

Since soils comprised of compact sand, gravel, or sand-and-gravel, are largely incompressible, the principal action at the tip of the pile is a lateral displacement of soil particles, a requisite for progressive penetration. There is little additional compaction of the soil. When driving ceases, the soil is already in a state of equilibrium, and the end resistance of the pile remains constant.

In compact sand, there is no significant reduction in intergranular space, and there is no increase in free water; thus, skin friction is not decreased by water lubrication during driving. After driving has stopped, the pressure of the sand grains against the pile is about the same as during driving, and the resulting skin friction is an important source of load-carrying capacity.

H-piles are especially suitable for use in these soils. Because of the high

A steel H-pile pulled after 14 years of service. The stiff clay core shows how tightly it was compressed, indicating the great amount of adhesion between the steel and the clay.

incompressibility of the soils, the comparatively small displacement resulting from driving a steel H-pile is sufficient to develop a high intensity of compressive stress both at the point and along the sides of the pile, with correspondingly high values for point resistance and skin friction.

H-piles in clay

The properties of clay are directly opposed to those of sand, gravel, and sand-and-gravel. Clay is distinguished from other soils chiefly by the smallness of the particles, the high degree of impermeability and compressibility, low coefficient of friction, and a cohesive strength that may vary from 100 lb per sq ft or less for very soft clay, to more than 4,000 lb per sq ft for very stiff or hard clay. These characteristics vary, generally speaking, with the moisture content. The engineering properties usually deteriorate as the percentage of moisture increases. This does not mean that the moisture content alone is a definite index to the properties of any clay. The strengths of two different clays having the same moisture content may differ considerably.

As we have noted, clay is compressible to a far greater degree than sand or gravel. When pressure is applied, the solid particles are pressed into closer contact with each other and water is squeezed out of the voids. The small amount of friction of the clay may generate only small frictional resistance during driving. This may further be reduced below its normal value by the lubricating action of the water squeezed out of the voids by compression at the pile point, and by lateral compression due to soil displacement and pile vibration under the action of the hammer.

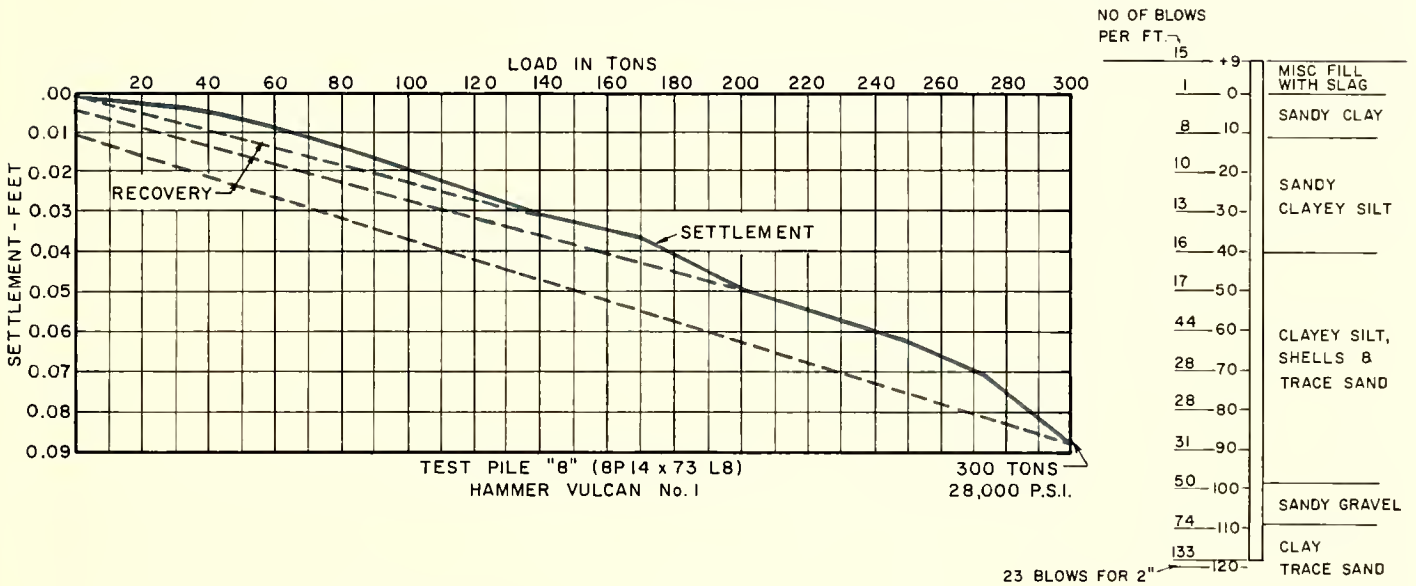
After driving is completed, however, the lateral pressure set up during driving forces the fine clay particles into close contact with the comparatively rough surface of the pile. This results in a strong adhesive bond that provides the mechanism for transferring load from the pile to the soil. In many clays, this bond is stronger than the shearing resistance of the soil. It is not unusual for the pile to come up with the spaces between flanges and web filled with cores of soil, and the flanges covered with adhered clay.

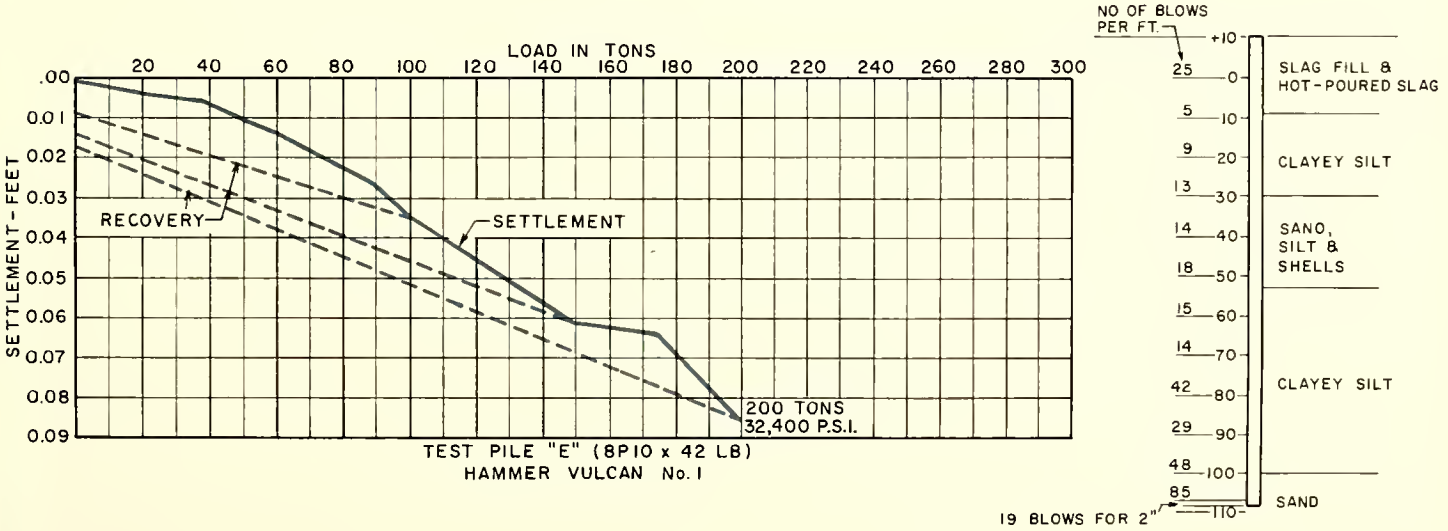
The great difference that may exist, in clay, between driving resistance and static load-carrying ability is well known. It is a common experience of pile drivers that a pile may drive easily but after a period of rest set up solidly. Upon redriving such a pile, the penetration per blow at the start of redriving is considerably less than the rate of penetration at the finish of the original driving. In the case of

H-piles, the static resistance to settlement almost always will be greater than the resistance during driving would indicate, because the skin friction increases from what may be a negligible value during driving to a substantial value after rest.

In hard, stiff clay containing a low percentage of voids and pore water, the compressibility will be small, and the amount of displacement and compression required to develop its full capacity will be correspondingly small. When an H-pile is driven into such clay, quite often the soil trapped between the flanges and web becomes so hard due to compression that it is carried down with it. The pile thereupon becomes in effect a displacement pile, and the core of soil trapped on each side of the web performs the same function of displacement that timber lagging serves in softer silts and clays. In such soils, plain H-piles will develop very satisfactory load-carrying values.

FIGURE 4A—H-PILE TESTS, "J" BLAST FURNACE FOUNDATIONS


LOAD IN TONS
20 40 60 80 100 120 140 160 180 200 220 240 260 280 300
SETTLEMENT - FEET
.00 0.01 0.02 0.03 0.04 0.05 0.06 0.07 0.08 0.09
RECOVERY
SETTLEMENT
TEST PILE "8" (8P14 x 73 LB)
HAMMER VULCAN No. 1
300 TONS
28,000 P.S.I.
NO OF BLOWS PER FT.
15 +9 MISC FILL WITH SLAG
1 0
8 10 SANDY CLAY
10 20
13 30 SANDY CLAYEY SILT
16 40
17 50
44 60 CLAYEY SILT, SHELLS & TRACE SAND
28 70
28 80
31 90
50 100 SANDY GRAVEL
74 110
133 120 CLAY TRACE SAND
23 BLOWS FOR 2"


LOAD IN TONS
20 40 60 80 100 120 140 160 180 200 220 240 260 280 300
SETTLEMENT - FEET
.00 0.01 0.02 0.03 0.04 0.05 0.06 0.07 0.08 0.09
RECOVERY
SETTLEMENT
200 TONS
32,400 P.S.I.
TEST PILE "E" (8P10 x 42 LB)
HAMMER VULCAN No. 1
NO OF BLOWS PER FT.
25 +10 0 SLAG FILL & HOT-POURED SLAG
5 10
9 20 CLAYEY SILT
13 30
14 40 SAND, SILT & SHELLS
18 50
15 60
14 70 CLAYEY SILT
42 80
29 90
48 100
85 110 SAND
19 BLOWS FOR 2"

Friction piles driven in problem soils

Conditions for which steel H-piles are particularly suited are sites where piles must be driven through original soil or fill containing stones, boulders, buried timbers, or very coarse gravel. This sometimes occurs where poorly specified fills are made for bridge abutments, or in old spoil areas along river banks. This is a common condition, for instance, on the shores of Manhattan Island, New York City. Under these conditions, the piles must withstand extremely hard driving in order to penetrate an upper stratum that often overlies softer layers that must be penetrated to reach the final bearing stratum. Steel H-piles are practically the only alternative to costly excavating or drilling methods.

In the remainder of this section, examples are given of the successful use of H-piles as friction piles, in soils classified as hard clay, stiff clay, and sandy clay.

Examples of H-piles used as friction piles

At our Sparrows Point plant, in connection with the 1955-56 expansion program, 30 piles were load-tested; some, to destruction. Since rock is at a depth of 700 feet, all of the piles are necessarily friction piles. Tests confirmed the opinion that H-piles would not fail until the steel was stressed to its yield point—between 35,000 and 40,000 psi. Failure was usually indicated by buckling of the steel at the top of the pile, with slight net settlement in the soil. Figure 4A shows a log of one of the piles loaded to 300 tons without failure, with a net settlement of only 0.12 inch. A working stress of 13,500 psi was established, corresponding to a design load of approximately 150 tons on a 14-in. x 73-lb pile. As 70,000 tons of H-piles were driven for this program, the high design stress utilized for these friction piles resulted in great economy.

Figure 4B shows a record of tests of H-piles driven for falsework in connection with the erection of the Bay Bridge, San Francisco. Note that piles are in 40 feet of water, and 13 feet of mud before reaching bearing material. Capacity was obtained by driving 40 to 45 feet into soil classified as clayey sand, very sandy silty clay, and finally silty clay. The load-settlement curves indicate that test loads varied from 100 tons minimum to 180 tons maximum.

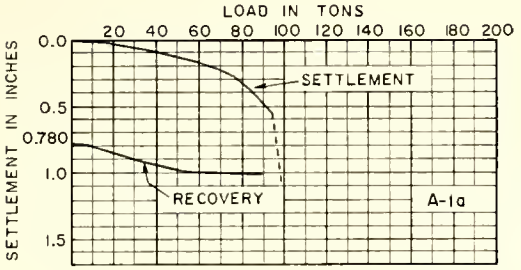
The Highway Research Board has published Special Report 67, *Records of Load Tests on Friction Piles.* This report documents the results of load tests on 412 piles of various types, under a wide variety of conditions. Most of the tests were not carried to failure. Table 4C is a record of the tests of 46 H-piles. The right hand column shows the net settlement, and it will be noted that with the exception of a few of the very heavily loaded piles, the net settlement is small.

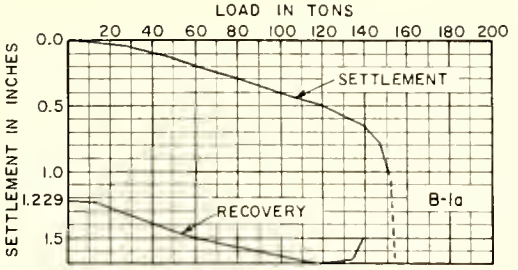
The fifth column shows the kind of soil in the final bearing stratum, providing dependable information concerning the types of soils for which H-piles can be successfully used as friction piles. The types of soil listed are sand, sand-clay, hard shale, hard clay, silt-and-sand, sand-and-gravel, fine sand, clay silt and sand, stiff clay, and sandy clay.

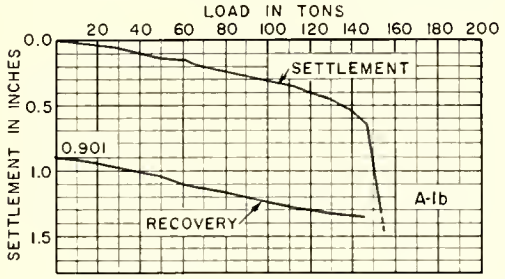
The last four test piles listed were part of the previously mentioned program at Sparrows Point, and are reported in Highway Research Board Report 36.

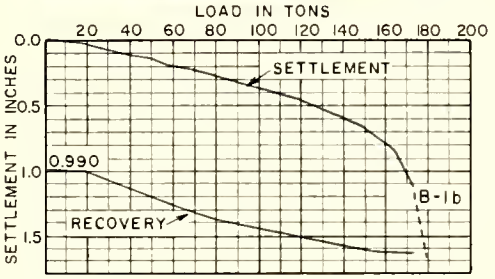
A careful study of these reports reveals no inferiority in the load-carrying capacity of H-piles used as friction piles, when compared to other types of piles. On the contrary, they show very satisfactory results when the load-bearing stratum is soil of good consistency.

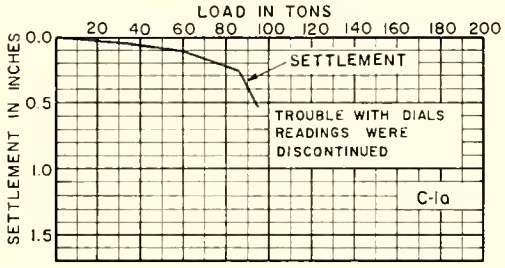
FIGURE 4B—H-PILE TESTS FOR BAY BRIDGE FALSEWORK, SAN FRANCISCO, CALIF.

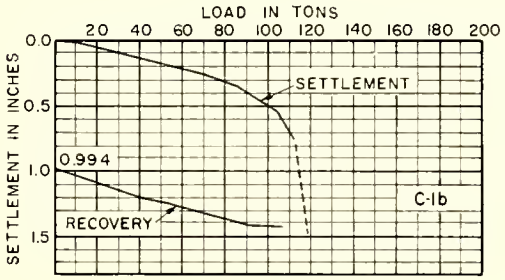




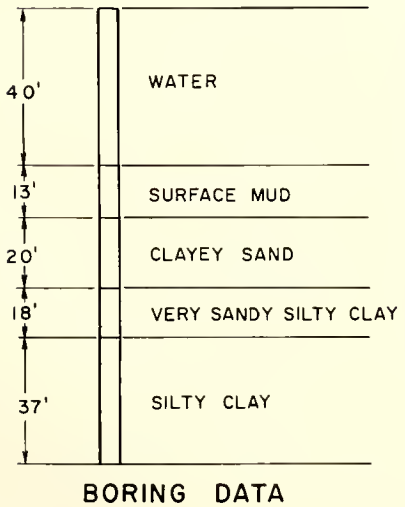








Driving Data			
Pile No.	*Total Length (feet)*	*Penetration (feet)*	*Penetration for the last blow (inches)*
A-la	99.5	44.7	0.131
A-lb	99.0	44.4	0.083
B-la	100.1	41.4	0.050
B-lb	99.6	40.4	0.056
C-la	99.2	43.2	0.364
C-lb	98.8	40.5	0.141



BORING DATA

No attachments for increasing the bearing capacity were used on these piles.

All piles were 12 in. x 65 lb wide-flange and were loaded with approximately 20 tons as falsework piles before testing.

Piles *A-la* and *A-lb* were in place from 25 to 28 days, and were released from falsework loading from 15 to 18 days before testing. *B-la* and *B-lb* were in place from 31 to 33 days, and had been released from 19 to 21 days. *C-la* and *C-lb* had been in place from 26 to 29 days under loading, and had been released from 21 to 24 days.

Piles *A-la* and *A-lb* were redriven 3 ft before testing. Piles *B-la* and *B-lb* were redriven 2 in. before testing. Piles *C-la* and *C-lb* were undisturbed before testing.

The load was applied in increments of 8.6 tons at intervals of 7.5 minutes. It was removed in increments of 17.3 tons at intervals of 4 minutes.

All piles were driven by a McKiernan-Terry No. 11-B2 double-acting steam hammer. Assumed rating: 19,000 ft-lb per blow.

All piles were pulled, and the pulling resistance measured. The frictional values in lb per sq in. of the full perimeter of the pile were as follows:

Maximum 7.08 lb per sq in.
Minimum 4.46 lb per sq in.
Average 6.01 lb per sq in.

TABLE 4C—FRICTION PILES—TABULATION OF LOAD TESTS ON H-PILES OBTAINED FROM SPECIAL REPORTS 67 AND 36, OF THE HIGHWAY RESEARCH BOARD.

Location	*H-pile Size, in.*	*H-pile Weight, lb*	*Length, ft*	*Final Bearing Stratum*	*Hammer Blow, ft-lb*	*Final penetration, in. per blow*	*Test Load, tons*	*Net Settlement, in.*
Barksdale Field, La.	12	53	60.5	Sand	15,000	0.42	100	0.07
Barksdale Field, La.	12	53	71.5	Sand	15,000	.33	100	0
Barksdale Field, La.	12	53	80	Sand	15,000	.35	100	.12
Barksdale Field, La.	12	53	62.4	Sand	15,000	.58	100	.13
Barksdale Field, La.	12	53	46	Sand	15,000	.48	100	.04
Barksdale Field, La.	12	53	55.4	Sand-Clay	15,000	.61	100	.09
Grand Ecore, La.	12	53	47	Sand	15,000	.25	100	.25
Grand Ecore, La.	12	53	54.4	Sand	15,000	.25	100	.20
Grand Ecore, La.	12	53	49.8	Sand	15,000	.27	100	.07
Galveston, Texas	12	53	25	Sand	15,100	na	134	.09
Port Huron, Mich.	12	72	90	Hard Shale	‡	na	160	0
Elko County, Nev.	10	42	33	Hard Gravelly Clay	30,000	.25	90	.03
Elko County, Nev.	10	42	22	Hard Gravelly Clay	21,000	.20	90	.05
Clark County, Nev.	10	42	95	Silt & Sand	6,750	.04	60	.25
Clark County, Nev.	12	53	25	Silt & Sand	6,750	.007	80	.02
Clark County, Nev.	12	53	26	Silt & Sand	6,750	.007	67	.02
Eureka County, Nev.	10	42	19	Sand & Gravel	13,200	.67	72	.28
Eureka County, Nev.	10	42	19	Sand & Gravel	15,750	.50	84	.23
Eureka County, Nev.	10	42	57	Sand & Gravel	15,750	.50	65	.03
Eureka County, Nev.	10	42	55	Sand & Gravel	15,750	.50	64	.02
Churchill County, Nev.	10	42	23	Fine Sand	15,000	.45	80	.03
Churchill County, Nev.	10	42	25	Fine Sand	15,000	.40	90	.15
Churchill County, Nev.	10	42	34	Fine Sand	15,000	.50	75	.20
Manitowoc, Wis.	12	53	32	Clay, Silt & Sand	8,750	.08	90	.18
Cleveland, Ohio	12	53	81	Stiff Clay	15,000	.13	118	.57
Cleveland, Ohio	12	53	105	Stiff Clay	15,000	.10	181	1.00
Cleveland, Ohio	12	53	97	Stiff Clay	15,000	.21	90	.80
Cleveland, Ohio	12	53	86	Stiff Clay	24,400	.29	90	.78
Cleveland, Ohio	12	53	101	Stiff Clay	24,400	.14	170	.60
Cleveland, Ohio	12	53	112	Stiff Clay	24,400	.27	130	.90
Cleveland, Ohio	12	53	32	Sand	15,000	.21	166	.45
Akron, Ohio	12	53	45	Sand-Silt-Clay	22,500	.17	150	.31
Lucas County, Ohio	14	73	86	Hard Clay	24,450	.03	150	2.10
Lucas County, Ohio	14	117	26	Hardpan	26,000	.014	300	.15
Lucas County, Ohio	14	117	49	Hard Clay	26,000	.017	300	.12
Washington, D.C.	12	53	27	Hard Clay	15,000	.14	100	.03
Washington, D.C.	14	73	26	Hard Clay	15,000	.10	100	.01
Washington, D.C.	12	53	26	Hard Clay	26,000	.033	100	.10
Washington, D.C.	12	53	65	Hard Clay	15,000	.13	100	.11
Washington, D.C.	12	53	46	Hard Clay	15,000	.14	100	.07
Brooklyn, Md.	14	102	93	Sand	15,000	.05	100	.11
Brooklyn, Md.	14	102	111	Sand	15,000	.13	100	0
Brooklyn, Md.	14	102	105	Sand	15,000	.09	100	0
Galveston, Tex.	14	73	48	Sandy Clay	13,100	.38	130	.08
Chelsea, Mass.	14	73	71	Sand	15,000	.10	120	.17
Chelsea, Mass.	14	73	99	Sand	15,000	.10	120	.06
Sparrows Point, Md.	10	42	140	Sand	15,000	na	220	*
Sparrows Point, Md.	12	53	140	Sand	15,000	.05	260	*
Sparrows Point, Md.	12	53	140	Sand	15,000	na	250	*
Sparrows Point, Md.	14	73	140	Sand	15,000	.06	354	*

**Recorded in Highway Research Board Report 36. These piles loaded to failure stress exceeded yield point of steel at 35/40 ksi. Friction between soil and piles was not exceeded.*

‡*Gravity hammer with ram weight = 3,855 lb.*

na—Information not available.

Ease of splicing and ease of handling in long lengths are two important advantages of steel H-piles.

95
95
90
95

Section 5

LATERAL SUPPORT FOR LONG PILES IN SOFT SOIL

For more than a hundred years it has been common practice in many harbors of the country to drive very long timber piles through deep mud to a satisfactory bearing stratum. In the past 25 years extremely long H-piles have been driven through very soft soil to refusal on rock or adequate resistance in firm soil. Their unquestioned adequacy has confirmed engineering opinion that any soil not a virtual fluid will provide adequate lateral support for a slender pile, regardless of length.

Table 5A shows a list of long test piles that obtained lateral support from mud or other soft soils. Extreme cases are Piles 1 and 2, for which a complete log of

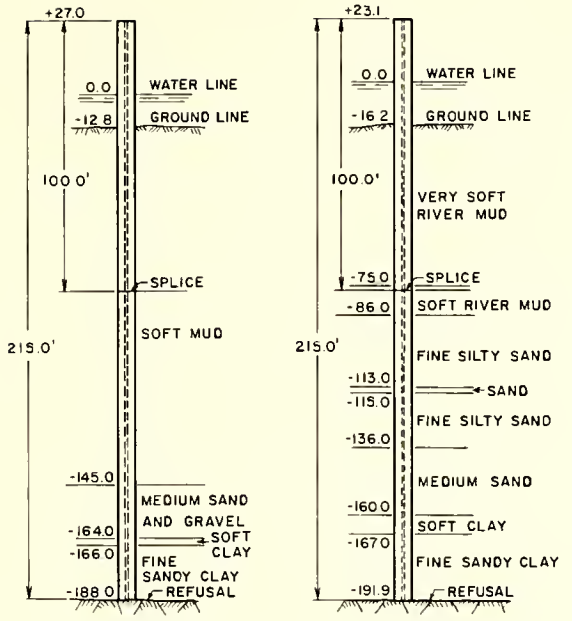
TABLE 5A—LOADING TESTS ON H-PILES LATERALLY SUPPORTED BY SOFT SOILS

Pile No.	*Location*	*Structure*	*Size of Pile and Weight per Foot, in. x lb*	*Length of Pile, feet*	*Length Unsupported and in Soft Material*	*Kind of End Bearing*	*Test Load, tons*	*Settlement, in.*		*Unit Stress Under Test Load, psi*
								Gross, Under Load	*Net, After Rebound*	
1	Pope's Creek, Md.	Potomac River Bridge	14 x 102	215	40 ft unsupported in water and air; 132 ft in soft mud. Total 172 ft.	Rock	200	0.93	0.24	13,330
2	Pope's Creek, Md.	Potomac River Bridge	14 x 102	215	39 ft unsupported in water and air; 70 ft in soft and very soft river mud; 27 ft in fine silty sand. Total 136 ft.	Rock	200	1.10	na	13,330
3	near Annapolis Md.	Chesapeake Bay Bridge	14 x 102	115	31 ft water, 29 ft semi-liquid organic silt.	Sand	200	0.63	0.02	13,330
4	Dearborn, Mich.	Press Shop	12 x 53	110	91.5 ft soft blue clay.	Rock	250	1.46	—	32,090
5	New York, N.Y.	Housing Project	14 x 89	110.3	18 ft of river mud and peat separated by 12 ft of sand from 50 ft of brown silt and gray clay in layers.	Rock	150	0.432	0.006	11,450
6	Sparrows Point, Md.	Proposed Plate Mill	14 x 73	141	57 ft river mud.	Gravel	140	0.70	0.156	13,050
7	Sparrows Point, Md.	Proposed Plate Mill	14 x 73	141	88 ft silt.	Gravel	150	0.852	0.456	13,980

n/a—information not available

FIGURE 5B—H-PILE TESTS FOR POTOMAC RIVER BRIDGE, POPE'S CREEK, MD.

Maryland Highway Dept.



Pile 1 **Pile 2**

Elevation of Pile Point, ft	*Penet. per Blow, in.*
— 63.0	①
— 108.5	②
— 111.0	6.00
— 114.6	2.00
— 117.6	2.40
— 122.6	2.80
— 130.6	2.70
— 134.6	2.60
— 142.6	2.10
— 146.6	1.90
— 148.6	0.85
— 149.6	0.78
— 151.6	0.70
— 154.6	0.55
— 156.6	0.70
— 157.6	1.00
— 159.6	0.98
— 161.6	0.90
— 163.6	0.85
— 165.6	0.80
— 169.6	0.70
— 174.6	0.65
— 175.6	0.70
— 180.6	0.65
— 182.6	0.70
— 184.6	0.60
— 188.0	Refusal

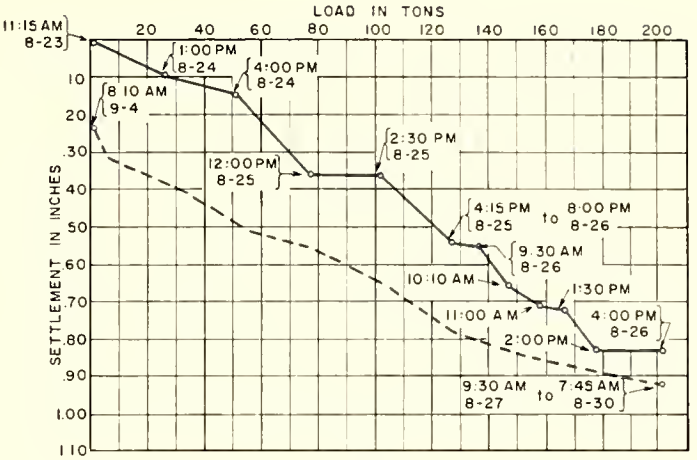
Pile 1

①*Pile sank under own weight.*
②*Pile sank under weight of hammer.*

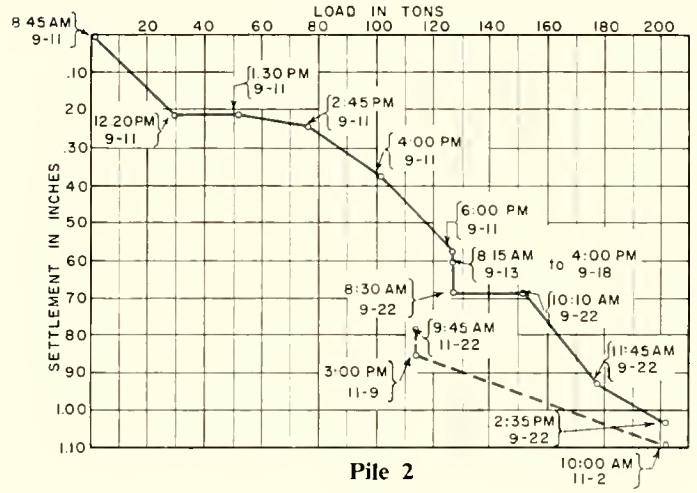
Elevation of Pile Point, ft	*Penet. per Blow, in.*
— 77.5	③
— 95.0	④
— 108.7	2.50
— 111.7	2.90
— 121.7	1.90
— 124.7	2.45
— 128.7	3.10
— 133.7	2.90
— 136.7	3.40
— 143.7	2.30
— 146.7	1.60
— 151.7	1.80
— 154.7	1.90
— 157.7	1.95
— 160.7	1.35
— 163.7	1.10
— 165.7	1.05
— 167.7	0.85
— 168.7	0.83
— 169.7	0.80
— 171.7	0.75
— 173.7	0.70
— 175.7	0.65
— 179.7	0.63
— 181.7	0.65
— 187.7	0.60
— 189.7	0.55
— 190.7	0.60
— 191.7	Refusal

Pile 2

③*Pile sank under own weight.*
④*Pile sank under weight of hammer.*



Pile 1



Pile 2

Piles were 14 in. Bearing Piles x 102 lb.
Piles were driven with a special McKiernan-Terry single-acting hammer with a 14000 lb ram and developing 37500 ft-lb per blow.
Load was applied by means of a loaded platform.
Readings were made with a level.

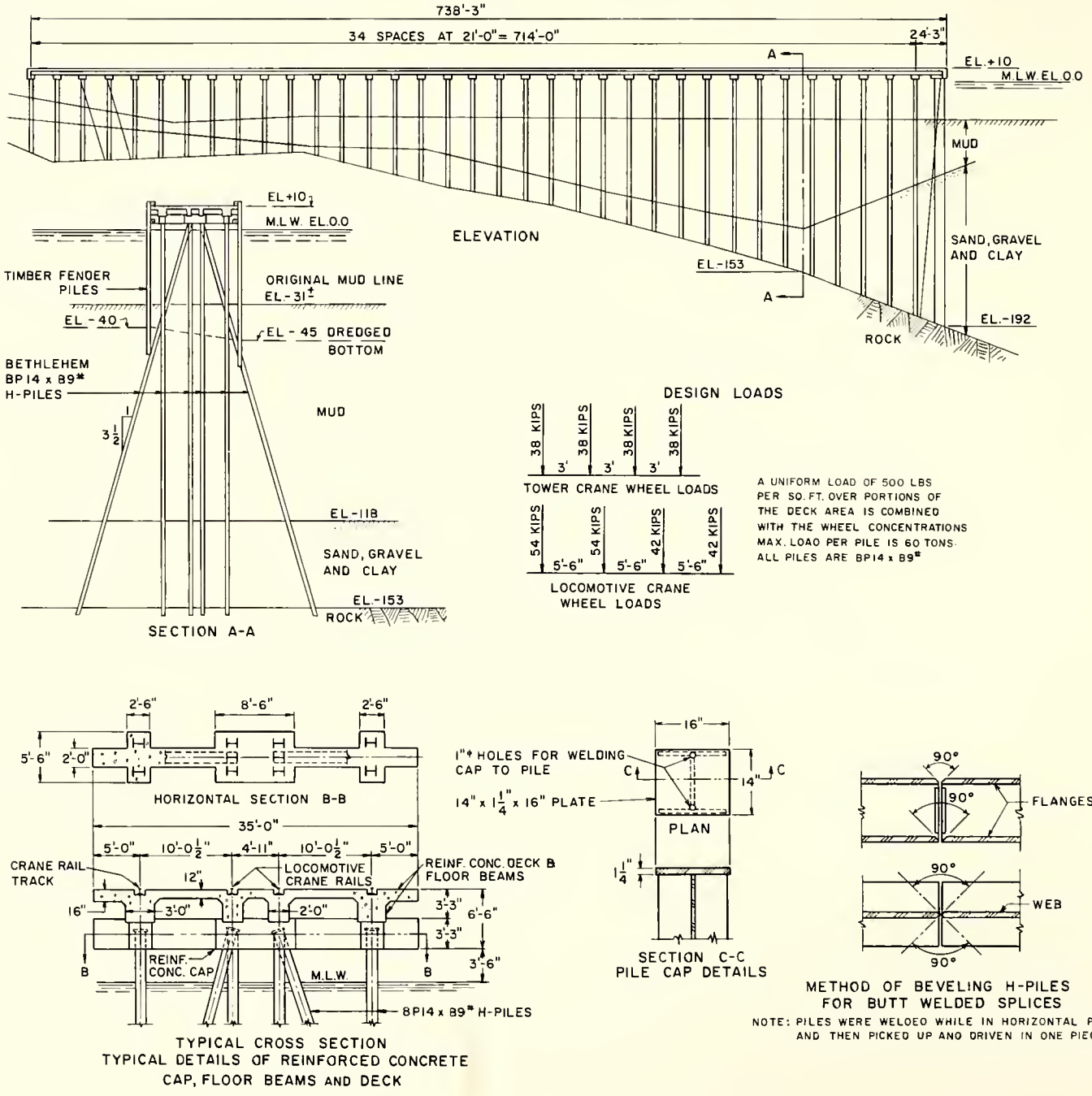
the tests is shown in Figure 5B. The top 40 ft of Pile 1 was in water and air, under which was 132 ft of soft mud. The top 39 ft of Pile 2 was in water and air, under which was 70 ft of soft mud. If the soft mud were assumed to provide zero lateral support, the unsupported length of 172 ft and 109 ft would result in $\frac{Kl}{r}$ values of 580 and 367. Refer to Section 6. However, Figure 5A shows that the gross settlement for Pile 1 under a test load of 200 tons was 0.93 in. with a net settlement after unloading of only 0.24 in. For Pile 2 gross settlement was 1.10 in. Since the piles performed so

well, it is a sound presumption that the mud has in fact provided adequate lateral support.

Another example, Pile 3 in Table 5A, is one of the test piles for the Chesapeake Bay Bridge. Here, there was 31 ft of water and 29 ft of soft organic silt through which the pile sank of its own weight. The pile was test loaded to 200 tons, with a gross settlement of 0.63 in., and net settlement after removal of load of only 0.02 in.

Another example is an outfitting pier at Hoboken, New Jersey, shown in Figure 5C. In this case the piles were as long as 200 ft; at its deepest point the mud extended to a depth of 118 ft below water level.

FIGURE 5C—OUTFITTING PIER, HOBOKEN, NEW JERSEY



Section 6

DETERMINING UNBRACED LENGTH

Suggested rules for determining the unbraced lengths of H-piles used in various combinations of soil stability are shown in Figures 6A to 6E, inclusive.

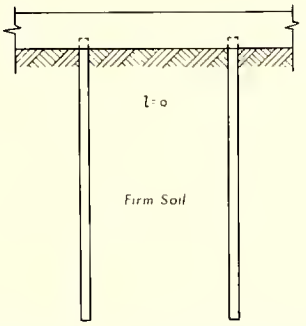


FIGURE 6A—*Entire length embedded in any soil or combination of soils, other than virtually fluid material.* In this case the pile is supported throughout its length, and no reduction in load is required because of slenderness ratio. See Section 5—*Lateral Support for Long Piles in Soft Soil.* Local Building Code regulations on allowable unit stress will apply.

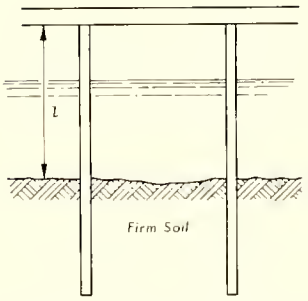


FIGURE 6B—*Unbraced Length Above Firm Soil.* The effective length should be taken as the distance from the surface of the firm soil to the nearest connection above the ground. *K = 1.0.*

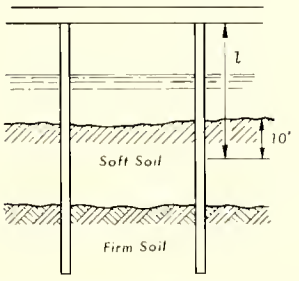


FIGURE 6C—*Unbraced Length Above Soft Soil.* The effective length should be taken as the distance from a point 10 feet below the surface of the soft soil to the nearest connection above the ground. *K = 1.0.*

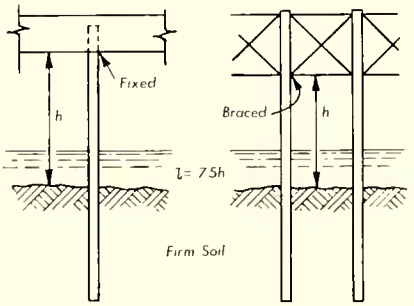


FIGURE 6D—*Unbraced Length, Top Fixed, Firm Soil.* Where the upper end of the unbraced length is fixed by embedment in concrete or is braced in a manner providing the equivalent of continuity, a K factor of 0.75 may be applied to the actual unbraced distance down to the surface of the firm soil, to allow for such restraint. *K = 0.75.*

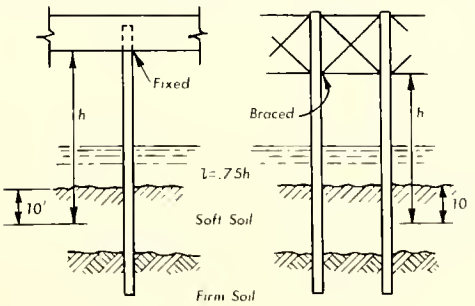


FIGURE 6E—*Unbraced Length, Top Fixed, Soft Soil.* This condition is the same as 6C, except that a K factor of 0.75 may be applied to the actual unbraced distance down to a depth of 10 feet below the surface of the soft soil, because of the fixed end condition. *K = 0.75.*

Table 6F gives the unit loads in tons and slenderness ratios for H-piles that are unsupported laterally for part of their length. They are computed in accordance with the provisions of the AISC Specification Sixth Edition, Section 1.5.1.3, for ASTM Specification A36. Loads above heavy lines exceed the maximum load recommended in Table 9A, page 33. In these cases, use the loads shown in Table 9A. Use the loads listed in Table 6F where these loads are shown below the lines. In other words, it is suggested that between the two tables, the lower value should be used.

TABLE 6F—SLENDERNESS RATIO AND LOADS IN TONS FOR H-PILES WITH PORTION OF LENGTH UNBRACED OR UNSUPPORTED.

Unbraced Length, Kl, ft	*BP 14*								*BP12*				*BP 10*				*BP 8*	
	117 lb		*102 lb*		*89 lb*		*73 lb*		*74 lb*		*53 lb*		*57 lb*		*42 lb*		*36 lb*	
	$\frac{Kl}{r}$	*Load, tons*	$\frac{Kl}{r}$	*Load, tons*	$\frac{Kl}{r}$	*Load, tons*	$\frac{Kl}{r}$	*Load, tons*	$\frac{Kl}{r}$	*Load, tons*	$\frac{Kl}{r}$	*Load, tons*	$\frac{Kl}{r}$	*Load, tons*	$\frac{Kl}{r}$	*Load, tons*	$\frac{Kl}{r}$	*Load, tons*
10	33.4	339	33.7	295	34.0	257	34.4	211	41.2	208	42.0	148	49.0	155	50.0	113	61.5	92
12	40.1	330	40.4	287	40.8	250	41.3	205	49.5	200	50.3	143	58.8	147	60.0	108	73.8	85
14	46.8	321	47.2	279	47.6	243	48.1	199	57.7	192	58.7	137	68.6	139	70.0	101	86.2	78
16	53.5	311	53.9	270	54.4	235	55.0	192	66.0	183	67.1	130	78.4	130	80.0	95	98.5	70
18	60.2	300	60.7	261	61.2	227	61.9	185	74.2	174	75.5	123	88.2	121	90.0	88	110.8	61
20	66.9	288	67.4	251	68.0	218	68.8	178	82.5	164	83.9	116	98.0	111	100.0	80	123.1	52
22	73.5	277	74.2	240	74.8	209	75.6	170	90.7	154	92.3	109	107.8	100	110.0	72	135.4	43
24	80.2	264	80.9	229	81.6	199	82.5	162	99.0	143	100.7	100	117.6	89	120.0	63	147.7	36
26	86.9	251	87.6	217	88.4	189	89.4	153	107.2	131	109.1	92	127.3	77	130.0	55	160.0	31
28	93.6	237	94.4	205	95.2	178	96.3	144	115.5	119	117.5	83	137.1	67	140.0	47	172.3	27
30	100.3	223	101.1	193	102.0	167	103.2	135	123.7	106	125.9	73	146.9	58	150.0	41	184.6	23
32	107.0	208	107.9	179	108.8	155	110.0	125	132.0	93	134.3	65	156.7	51	160.0	36	196.9	20
34	113.6	192	114.6	166	115.6	143	116.9	115	140.2	83	142.7	57	166.5	45	170.0	32	209.2	
36	120.3	176	121.3	151	122.4	130	123.8	104	148.5	74	151.0	51	176.3	40	180.0	28	221.5	
38	127.0	159	128.1	137	129.2	117	130.7	94	156.7	66	159.4	46	186.1	36	190.0	26	233.8	
40	133.7	144	134.8	123	136.0	106	137.5	85	164.9	60	167.8	41	195.9	33	200.0	23	246.2	
42	140.4	130	141.6	112	142.8	96	144.4	77	173.2	54	176.2	37	205.7		210.0		258.5	
44	147.1	119	148.3	102	149.6	87	151.3	70	181.4	49	184.6	34	215.5		220.0		270.8	
46	153.8	109	155.1	93	156.4	80	158.2	64	189.7	45	193.0	31	225.3		230.0		283.1	
48	160.4	100	161.8	86	163.2	73	165.0	59	197.9	41	201.4		235.1		240.0		295.4	
50	167.1	92	168.5	79	170.0	68	171.9	54	206.2		209.8		244.9		250.0		307.7	
52	173.8	85	175.3	73	176.8	63	178.8	50	214.4		218.2		254.7		260.0		320.0	
54	180.5	79	182.0	68	183.6	58	185.7	46	222.7		226.6		264.5		270.0		332.3	
56	187.2	73	188.8	63	190.4	54	192.6	43	230.9		235.0		274.3		280.0		344.6	
58	193.9	68	195.5	59	197.2	50	199.4	40	239.2		243.4		284.1		290.0		356.9	
60	200.6		202.2		204.0		206.3		247.4		251.7		293.9		300.0		369.2	

Section 7

TENSION PILES

There are a number of applications in which H-piles are used in tension. These include A-frame type anchorages for sheet pile bulkheads, bulkheads where a tension pile is connected directly to the waler in lieu of a conventional tie rod anchorage, and transmission tower foundations.

As tension piles are actually a form of friction pile, the amount of tension that can be developed depends upon the consistency of the soil. For clays, the ultimate resistance can be estimated by multiplying the perimeter of the rectangle enclosing the cross section of the pile by the embedded pile length, and then multiplying this area by the average shear resistance of the soil. A liberal safety factor should be applied to this ultimate value, preferably not less than 2.5.

Where the tension pile is driven vertically into sand, a conservative ultimate tension value can be estimated by computing the active soil pressure against the area of the pile determined as above, and multiplying this total pressure by a coefficient of friction of about 0.35. Again, a safety factor of 2.5 should be used for design.

Where the tension member of an A-frame is driven on a batter, the weight of the soil overlying the anchorage will add to the pulling resistance.

If there is doubt concerning tension value, it is advisable to conduct an extraction test. Figure 7A shows such a test to determine the value of the tension piles in an A-frame anchorage for a dock at Lackawanna, New York. Here, as the boring in Figure 7B shows, the soil was largely clay. Penetration tests had shown that for a considerable portion of the depth the sampling spoon could be pushed, except in the upper sand stratum, and the clay-gravel stratum overlying rock. The boring information was not reassuring, and it was considered imperative to conduct an extraction test.

Although the tension piles in the structure were to be driven on a batter, the test pile was driven vertically, for convenient operation. Figure 7B shows the load deflection diagram for the test pile. Failure load was 282 kips, a very satisfactory result which could hardly have been predicted from the boring data.

Figure 7A also shows the apparatus used for the test. Three piles were driven in line, with the pile to be tested in the center. The tension was applied by a large jack pushing upward on a yoke which was attached to the test pile. The reaction was taken by the two exterior compression piles. A dial gage measured movement of the test pile in thousandths of an inch.

FIGURE 7A—H-pile being tested by means of the anchor-pile method.

FIGURE 7B—H-PILE EXTRACTION TEST

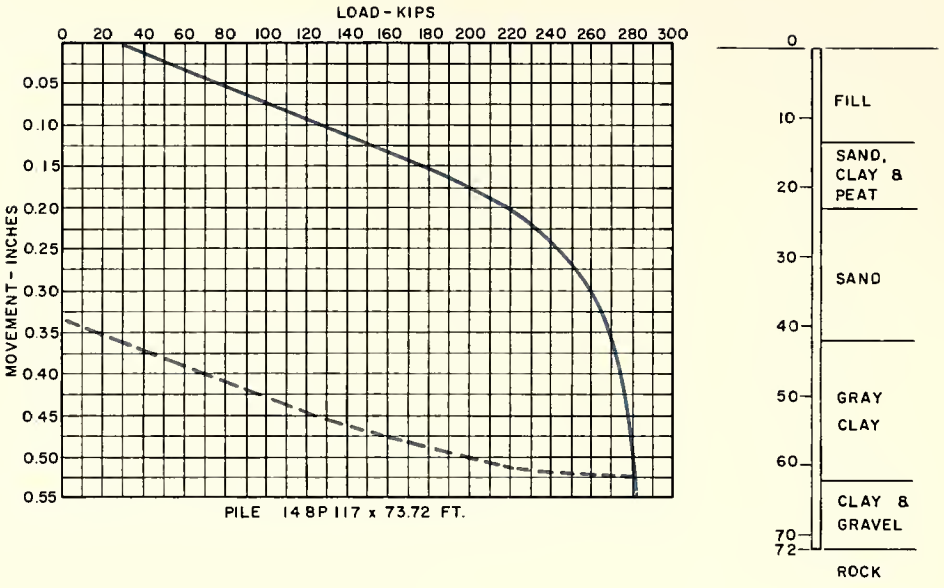


FIGURE 7C—DOCK WALL FOR LACKAWANNA SHIP CANAL—WEST SIDE

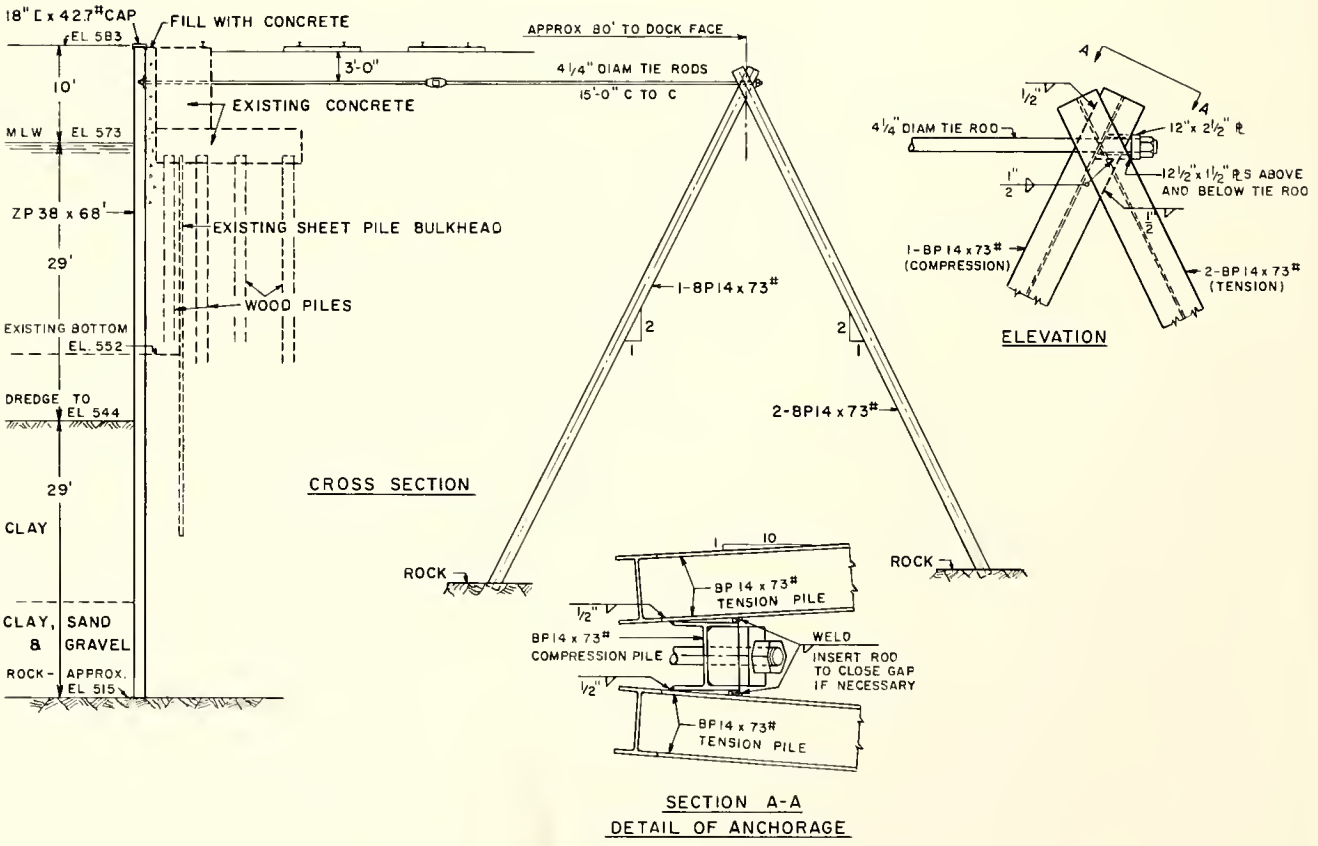


Figure 7C shows a detail of the A-frame anchorage in the structure. Anchorages were on 15-ft centers, and were comprised of one compression pile and two tension piles, all driven to rock. The tension piles were driven on a batter, in two directions. Had they been driven parallel at such close spacing there would have been a probability of excess overlapping of stresses in the small amount of soil between them. The three piles were welded together at the top.

Another example of tension piles is shown in Figures 7D and 7E, where the piles were driven 45 degrees to the horizontal, to serve as an anchorage for a Z-pile retaining wall. The local building code permitted only 20 tons for each pile in tension. As a design load of 60 tons per pile was required, this limitation would have made the design impractical. A tension pile test was decided upon.

Three piles were driven at 45 degrees in a location where they could be incorporated in the structure. Figure 7D is a photograph of the test. The two exterior piles were used as reaction piles, and functioned in compression. A yoke between them served as a transfer bridge for the two jacks, in exerting a pull on the test pile through the attachments shown in the photograph.

Figure 7E shows a cross section. Since the structure was to be a retaining wall, the soil above the failure plane was not considered available for friction on the tension pile. For the test, a hole was bored in the soil for a depth of 30 feet as shown in Figure 7F, the hole being somewhat smaller than the H-pile, so that the edges of the flanges gripped the soil. The test pile was inserted in the hole, and driven for 26 feet into the underlying soil beyond the bottom of the bored hole. The ultimate pulling resistance of the pile was determined, as indicated in the load deflection diagram of Figure 7F. A load of 300 tons resulted in a net movement of only 0.40 inch after removal of the load, indicating that the pile provided a large safety factor for the 60-ton design load.

FIGURE 7D—TEST OF AN H-PILE, DRIVEN ON THE BATTER, FOR USE AS A TENSION PILE.

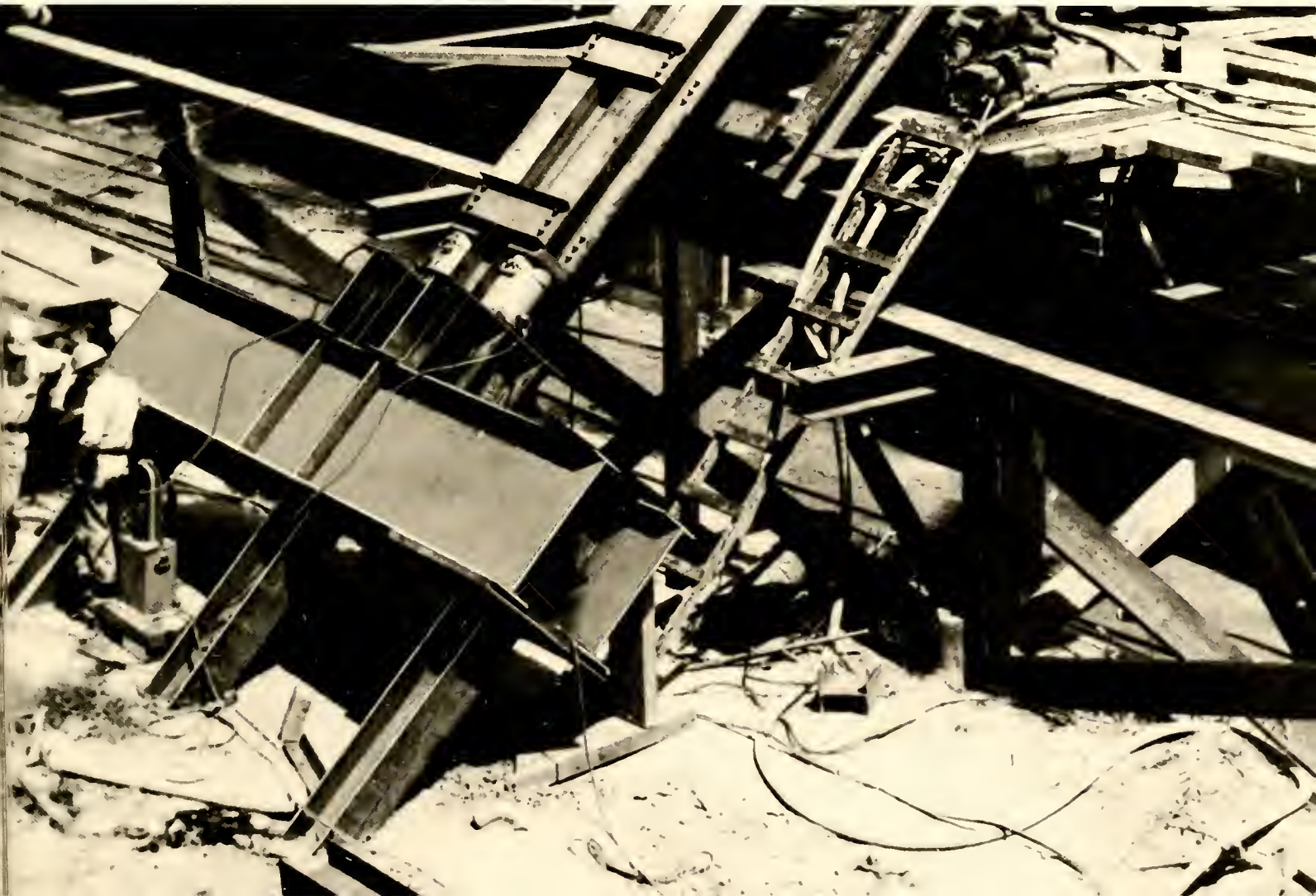


FIGURE 7E—SHEET PILE RETAINING WALL, WEST END OF BLOOMING MILL

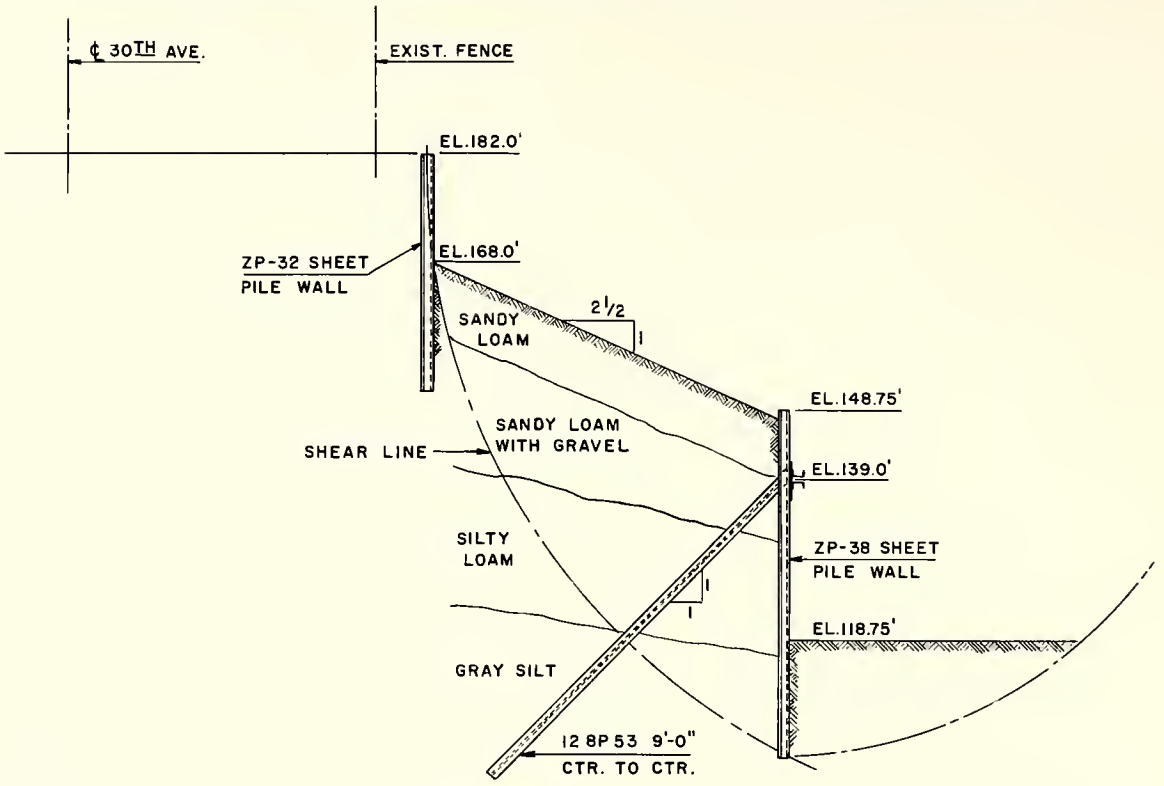
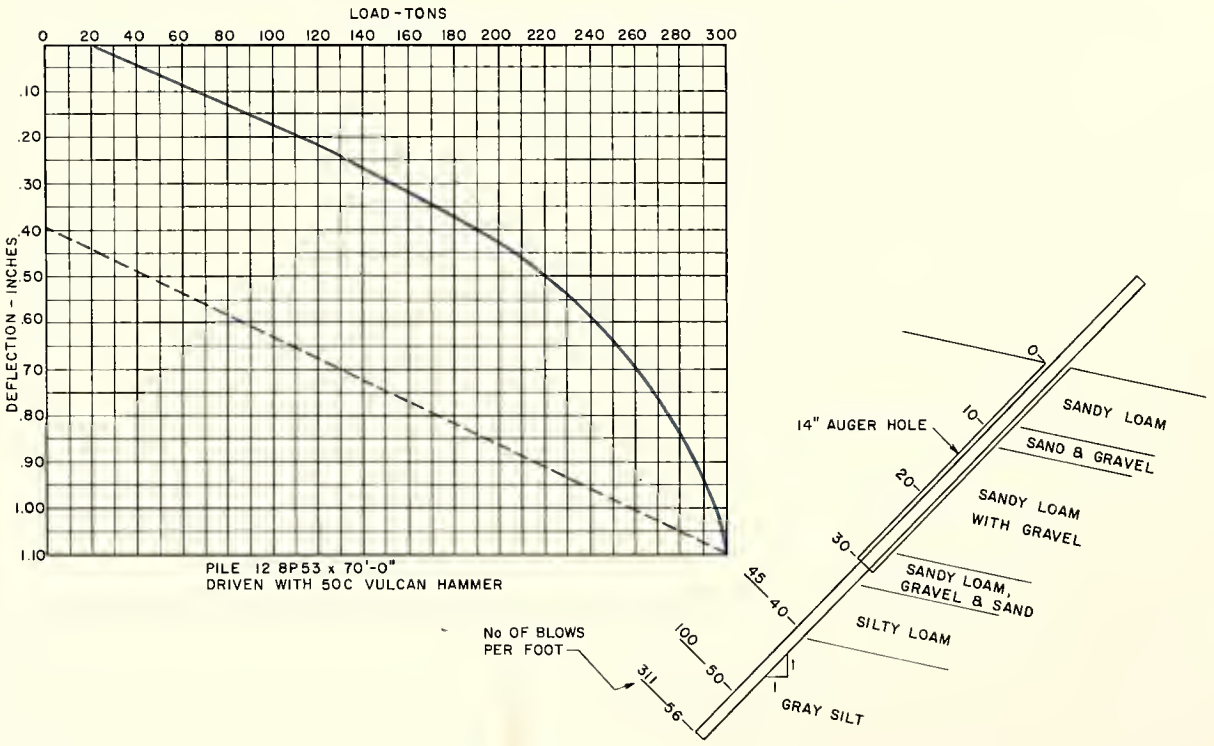


FIGURE 7F—TENSION PILE FAILURE LOAD TEST



THE LIFE OF STEEL H-PILES

H-piles buried in earth

Since steel piles were first used in the late 1800's, their service life has far exceeded a purely theoretical estimate of performance, especially in soils where adverse conditions and chemicals have been present. Of literally tens of thousands of installations, not a single report of corrosion causing the failure of a structure has been brought to our attention.

Despite the lack of evidence showing failures caused by corrosion, some engineers have tended to rely upon a strictly theoretical approach to the subject of corrosion. Soils are analyzed to determine such factors as acidity or alkalinity (pH), resistivity, and chemical constituents. An acid pH, low resistivity, or the presence of chlorides or sulphides in the soil are considered to be a corrosive environment, precluding the use of steel piles, or requiring expensive protective expedients.

Thus, although the statistical record is perfect, there have been cases where the theoretical appraisal was permitted to dominate, and steel piles were made more costly for the client than conditions warranted. Or, for such an untenable reason, the use of the better pile was denied.

Until 1961, no comprehensive research effort was made to ascertain the disparity between theoretical and actual performance. In that year, the National Bureau of Standards initiated a continuing program of field research. The first report on this program is available in National Bureau of Standards Monograph 58, *Corrosion of Steel Pilings in Soils*, by M. Romanoff. It can be obtained from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402, or from the nearest Bethlehem Steel sales office.

The monograph reports on the extent of corrosion on steel piles in service up to 40 years in various structures and many different soil environments. The author summarizes his findings as follows:

"Steel pilings that have been in service in various underground structures for periods ranging between 7 and 40 years were inspected by pulling piles at 8 locations and making excavations to expose pile sections at 11 locations. The conditions at the sites varied widely. The soil types ranged from well-drained sands to impervious clays; soil resistivities ranged from 300 ohm-cm to 50,200 ohm-cm. Soil pH ranged from 2.3 to 8.6.

"The data indicate that the type and amount of corrosion observed on the steel pilings driven into undisturbed natural soil, regardless of the soil characteristics and properties, is not sufficient to significantly affect the strength or useful life of pilings as load-bearing structures.

"Moderate corrosion occurred on several piles exposed to fill soils which were above the water table level or in the water table zone. At these levels the pile sections are accessible if the need for protection should be deemed necessary.

"It was observed that soil environments which are severely corrosive to iron and steel buried under disturbed conditions in excavated trenches were not corrosive

to steel pilings driven in the undisturbed soil. The difference in corrosion is attributed to the differences in oxygen concentration. The data indicate that undisturbed soils are so deficient in oxygen at levels a few feet below the ground line or below the water table zone, that steel pilings are not appreciably affected by corrosion, regardless of the soil type or the soil properties. Properties of soils such as type, drainage, resistivity, pH or chemical composition are of no practical value in determining the corrosiveness of soils toward steel pilings driven underground. This is contrary to everything previously published pertaining to the behavior of steel under disturbed soil conditions. Hence, it can be concluded that National Bureau of Standards data previously published on specimens exposed in disturbed soils do not apply to steel pilings which are driven in undisturbed soils."

Figures 8A to 8D, inclusive, are photographs of several of the piles that were inspected by Mr. Romanoff and covered in his report. The captions are Mr. Romanoff's. Note that they had been in service for 40, 12, 37, and 7 years, respectively.

At our Sparrows Point plant, H-piles have been used for approximately 25 years for the foundations of all important structures. During the major expansion of this plant in 1955 and 1956, field investigation was made of H-piles that had been in place for up to 15 years. Some piles were excavated in test pits; others were uncovered in routine construction operations. In all, H-piles were inspected in some 40 locations. Some of the results of these investigations are shown and described in Figures 8E through 8H.

FIGURE 8A—Sandblasted 3-ft section from the 40-year-old piling extracted from an abutment wall in the Corps of Engineers Dam and Lock No. 8 on the Ouachita River near El Dorado, Arkansas.

The section was exposed about 18 ft below the ground line and it is the only portion of the pile which contained pits of measurable depth. The maximum pit was 26 mils in depth.

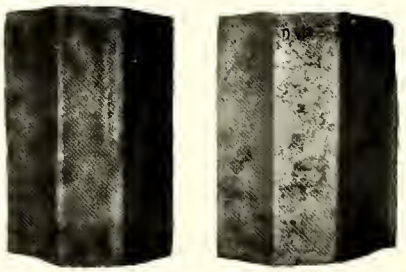


FIGURE 8B—Sections (1.5 ft by 1 ft) cut from a piling which was pulled from the north upstream wingwall of the Grenada Dam Spillway at Grenada, Mississippi after exposure for 12 years.

Sections were cleaned by sandblasting.

D103A, section of pile exposed to fill soil.

D103B, section of pile exposed to natural soil.

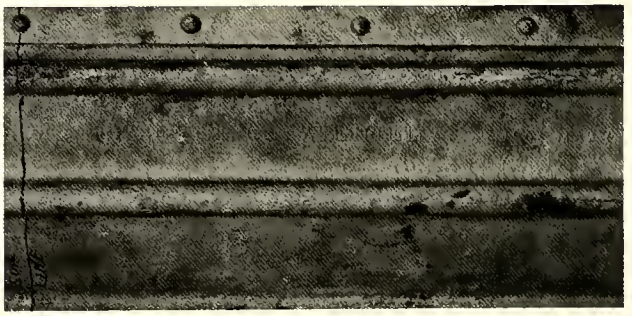


FIGURE 8C—A 3-ft section of steel sheet piling exposed below the soil line in a cofferdam structure in the Lumber River near Boardman, North Carolina.

Exposure, 37 years.

FIGURE 8D—Sandblasted sections of Z-type sheet piling cut from two different locations in the Vicksburg Floodwall after exposure for 7 years.

Although cinders were present in the soil at both locations, no significant corrosion occurred.

B101, Section removed from floodwall at station 16+32.

B102, Section removed from floodwall at station 23+83.

FIGURE 8E—A pile that had been in place under the 68-inch strip mill for 10 years with no protective coating. Note the bottom of the concrete cap, at the top of the picture; this was at elevation 0, ground water level. It was found to be in perfect condition, without any corrosion. The mill scale was still intact on the steel.

FIGURE 8F shows a pile that was under a sulphate storage building. The bottom of the concrete cap was at elevation +1.0, one foot above ground water level. Before driving, the pile had been coated with a prime coat of Bitumastic, and then a coat of hot enamel, from elevation +1.0 to −2.0. The coating was intact, although it had been driven through abrasive slag. The pile was in perfect condition, both where coated and below the coated area.

FIGURE 8G—This uncoated pile was under a blast furnace gas washer. The bottom of the concrete slab was 4.5 feet above ground water level. The pile was in perfect condition below water level, but showed some pitting above water level, with a maximum pit depth of .02 inch. The loss of cross section was inconsequential.

FIGURE 8H—Sections of the 139-ft H-piles pulled from Sparrows Point, Maryland, after exposure for 18 years. Left, water table zone consisting of fill material; center, clay soil stratum at about elevation —30 ft; and right, coarse sand and gravel stratum underlain by clay between elevations —110 and —126 ft. The pile was cleaned by sandblasting. Note the excellent condition of the butt weld at the splice in the center photograph.

Figure 8H shows sections of piles 139 feet long which had been in place 18 years, and were pulled in connection with the investigation of Mr. Romanoff for the National Bureau of Standards. The caption under the photograph is quoted from his report.

H-piles immersed in fresh water; atmospheric exposure over fresh water; atmospheric exposure over earth (except along the shore adjacent to sea water)

All these types of exposure are entirely favorable. Many bridge structures built 50 to 60 years ago in the State of Nebraska were supported by steel piles. In a paper presented before the American Society of Civil Engineers in 1932 and published in the September 1932 issue of *Civil Engineering* by J. C. Mason, Bridge Engineer, on the subject of steel pile foundations, the following statement was made in reference to these steel piles: "Of a great many such piles examined, it was estimated that the decrease in section from the original has not been more than 1 per cent in 20 years." Many of these structures were of the "bedstead" type, where the abutment piles extended above the ground and were exposed to the atmosphere, and the pier bents extended above water to directly support the superstructure. Consequently the piles were exposed to soil, to air, and to fresh water. We are informed by the present Bridge Engineer that at this writing, more than 30 years after the above-mentioned article was written, most of these structures are still in service.

The examination of part of the steel piling under a bridge near Enid, Oklahoma, after about 34 years, showed the shop coat of paint was still intact about 6 inches under the ground and the piling was in excellent condition. A little rust streak occurred at the line of contact between ground and air. The report states that not even 1 percent of the section of the piling had been lost due to corrosion.

Figure 8J, below, shows a sheet piling dock at Black Rock, near Buffalo, New York. It was built in 1910. The photograph, taken in August, 1963, shows the dock still in service, in excellent condition.

FIGURE 8J—This steel sheet piling dock at Black Rock, near Buffalo, N. Y., was built in 1910. It was still in excellent condition in August, 1963, when this photograph was taken.

H-piles in sea water

The use of steel H-piles in sea water has become increasingly common, and the questions of corrosion and appropriate methods of protection merit serious study. The life of unprotected steel piling in sea water installations varies with the conditions of exposure. Wharf or pier construction in protected harbors can be expected to have a considerably longer life than shore structures which are subject to salt spray, wave action, and sand abrasion.

Unprotected steel in sea water corrodes by an electrochemical process. Sea water having a low electrical resistance, functions as an electrolyte. Certain areas of the steel are anodic, and current flows from them through the sea water to other, cathodic, areas. The circuit is completed through the metal. Corrosion occurs only at the anodic areas, and if the flow of current is prevented, corrosion cannot occur.

In waterfront structures actually standing in sea water, the area immediately above water level—the so-called splash zone—is wet with a thin film of water that is saturated with oxygen. The area below water level is, by comparison, oxygen-starved, and is anodic to the splash zone. The most severe corrosion usually occurs within a zone about four feet high, extending from two feet below to two feet above low-water level. This is the area that is either continually or almost continually anodic as the tide rises and falls.

The rate of corrosion decreases rapidly with water depth, and is comparatively low at depths greater than two feet below the low-water level. In some cases there is an increase at the mud line, not usually serious.

Cathodic protection

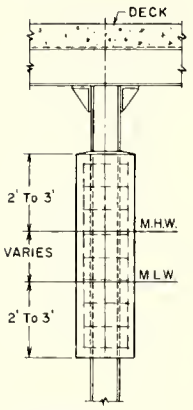
This process suppresses the flow of current from anodic to cathodic areas of the steel, and thereby prevents corrosion. Anodes are placed at proper intervals in the electrolyte (the sea water) and are connected electrically to the steel to be protected. The most common method uses inert graphite or Duriron anodes connected to the steel through a rectifier or other source of direct current to maintain a higher potential in the anode and ensure a flow of current to all areas of the immersed pile. Another method uses sacrificial anodes of magnesium or zinc, with galvanic corrosion of the anodes supplying the protective current. The choice between the two methods depends upon economic factors.

Cathodic protection effectively prevents corrosion in the submerged portion of the steel, and up to what is approximately the half-tide level. In an H-pile structure where one anode protects a number of piles, as in a pier bent, the piles must be connected electrically. A very small amount of current is required. When a system is first installed, a current of 5 to 10 milliamps per square foot of exposed surface is usually required, and will result in a white calcareous deposit on the steel. The amount of current required decreases thereafter, and may become as little as 1 milliamp per square foot. It is desirable to maintain the current so that the steel has a negative potential of 0.85 volts or more, with respect to a saturated $Cu\text{-}CuSO_4$ reference electrode. Corrosion of steel does not occur at this potential.

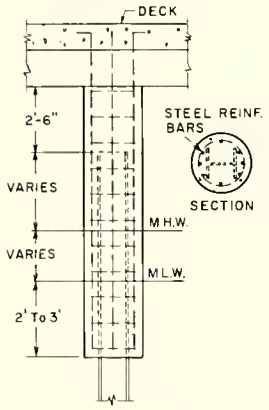
Protective coatings

When the steel is not cathodically protected, a protective coating reduces corrosion as long as the coating lasts. Deterioration occurs first in the tidal zone, the area most exposed to all forms of attack, and progresses upward to the top and, at a slower rate, downward through the immersed zone. Maintenance of the coating in the tidal range can be expensive, and in many cases, virtually impossible.

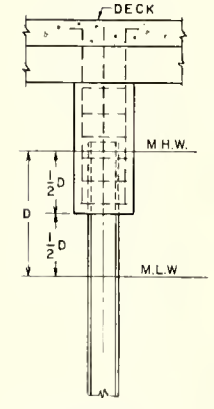
FIGURE 8K—CONCRETE JACKETS FOR STEEL H-PILES



(A) PARTIAL JACKET



(B) FULL JACKET



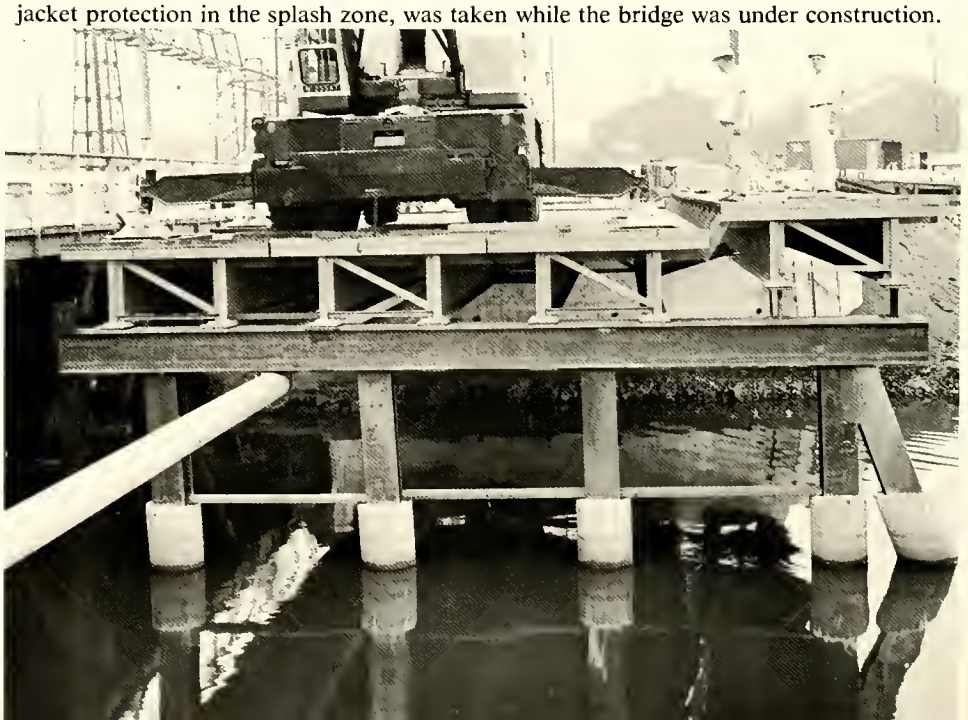
(C) PARTIAL JACKET WITH CATHODIC PROTECTION

One significant protection method is a coating of flame-sprayed aluminum that is applied to specially prepared steel piles. Test piles driven in 1950 still show no corrosive deterioration, even in the splash zone. Tidal testing since that time reflects no evidence of corrosion of the steel. Unprotected control piles, in contrast, show a loss in thickness of 0.04 in. per year in the splash zone, and 0.004 in. in the submerged area. While no really long-term test results are as yet available, these interim test reports give strong indication that this coating will outperform even the best low-alloy steel piling records. And, in many applications, the reduction in both initial labor and subsequent maintenance offers considerable advantage over the traditional organic and inorganic coatings, and concrete jacketing.

Concrete jacketing of steel piling has been very effective. It should extend from two or three feet below low-water level to two or three feet above high tide. With this protection, it is unlikely that cathodic protection will be necessary, but may be installed at a later time if required. Where cathodic protection is part of the original installation, the concrete jacket need only extend down to the mid-tide level. Figure 8K shows three methods of protection. Dense concrete should be used, and no steel reinforcement should be placed within four inches of the surface of the concrete.

A proprietary coating, consisting of a blend of epoxy and polyamide resins, is available. It is claimed that this coating provides successful protection for the splash zone on sheet piling and bearing piles subject to exposure in salt water. The compound is a heavy paste, to be trowelled on or applied by hand in excess of ⅛-inch thickness. It cures under water, as well as in the dry, to a hard, tough, tightly adherent corrosion-resistant coating. Application of the coating, and repairs to it, can be made underwater by a person wearing a skin-diving suit and rubber gloves. The compound is easily molded to the thickness required, either above or below water, if gloves are kept wet and the mixture is viscous enough while curing to resist removal by the washing action of waves.

This view of Humphrey's Creek Bridge, Sparrows Point, Maryland, showing the concrete jacket protection in the splash zone, was taken while the bridge was under construction.



Section

9

SAFE LOADS AND DESIGN DATA FOR BETHLEHEM H-PILES

The amount of load that can be placed on an H-pile is closely related to the type of soil into which it is driven. A steel pile driven through any soil to hard rock will not fail until the yield point of the steel is reached. This is also generally true for compact, well-graded sand, or sand-and-gravel, when the pile is driven to refusal. For the fine loose sands, silts, and clays, no general statement can be made, since the particular soil, rather than the pile, may be the limiting factor. The load to be carried by the individual pile should be determined by one of the approved methods, preferably by loading tests, although pile-driving formulas are very commonly used. See Section 13, *Pile-driving Formulas*, page 50.

Unit stresses

The unit stress for H-piles is usually set at a conservative percentage of the stress allowed for structural steel columns. It is current practice, in many building codes, to allow a maximum unit axial stress of 12,000 psi, which is less than two-thirds of the unit stress allowed for an equivalent structural steel column in a building. This stress provides an adequate factor of safety to cover the particular contingencies associated with pile installation and service. This subject is fully discussed in the bulletin, *Pile Foundations*, published by the American Iron and Steel Institute, 150 East 42nd St., New York, New York 10017. Copies of this bulletin are available upon request.

End-bearing piles

It is recommended that the load per pile be limited to a maximum unit stress of 12,000 psi for piles driven to hard rock or medium hard rock, provided that each pile is driven to practical refusal at the elevation at which the borings indicate rock. Driving to refusal is discussed on page 45. Under these conditions, it is suggested that load tests are not necessary, although opinions vary on this point.

Many load tests have proved that when an H-pile is driven to refusal on rock, load at failure will correspond to the yield point of the steel in the 35,000 to 40,000 psi range, and therefore a generous factor of safety is assured where the economy of the 12,000 psi unit stress has been utilized.

TABLE 9A

Section Number	*Wt per Ft, lb*	*Load, tons**
BP 8	36	63
BP 10	42	74
BP 10	57	100
BP 12	53	93
BP 12	74	130
BP 14	73	129
BP 14	89	157
BP 14	102	180
BP 14	117	206

**Loads shown correspond to an allowable unit stress of 12,000 psi.*

When an H-pile is driven to end-bearing in soft rock, hardpan overlying rock, or compact sand-and-gravel overlying rock, it can ordinarily be driven to practical refusal. Under such conditions, maximum loads equal to those recommended above for H-piles driven to rock are warranted. However, test loads are recommended, particularly for piles designed for heavy loads, since the conditions here are not so definite as they are when H-piles are driven to hard or medium hard rock. For a detailed discussion, see Section 3, *End-Bearing Piles.*

Minimum spacing of end bearing piles

To find the minimum spacing of piles, when driven to rock, it is first necessary to find the required area of distribution for each pile load. This is done by dividing the load value of the pile by the presumptive bearing capacity of the rock. The square root of the area of distribution for each pile is then the minimum spacing between piles, assuming equal spacing in both directions. That is,

$$\sqrt{\frac{\text{load value of the pile}}{\text{presumptive bearing capacity of the rock}}} = \text{minimum spacing per pile}$$

Example: load value of the pile =100 tons
bearing value (soft rock)=8 tons per sq ft

$$\sqrt{\frac{100}{8}} = 3.5 \text{ ft (approx)}$$

There are two limitations to the use of this formula, in order to provide for deviations in driving. First, that the minimum spacing (center-to-center) should not be less than two ft, and second, the spacing should not be less than 1.75 times the diagonal dimension of the pile. Either limitation will usually govern spacing where piles are being driven to hard rock having a bearing value beyond that which can be developed with piles.

Example:
load value of the pile =100 tons
bearing value
(med. hard rock) = 40 tons per sq ft

$$\sqrt{\frac{100}{40}} = 1 \text{ ft-7 in.}$$

As this is less than 2 ft, the minimum spacing rule applies. For a 14-in. pile, the minimum spacing will be approximately 2 ft-10 inches.

There has been considerable discussion concerning the high concentration of

TABLE 9B—MAXIMUM BEARING CAPACITIES OF ROCK AND SOIL

Class	*Material*	*Capacity,* tons per sq ft*
1	Hard sound rock	60
2	Medium hard rock	40
3	Hardpan overlying rock	12
4	Compact gravel and boulder-gravel formations; very compact sandy gravel	10
5	Soft rock, not disintegrated	8
6	Loose gravel and sandy gravel; compact sand and gravelly sand; very compact sand-inorganic silt soils	6
7	Hard dry consolidated clay	5
8	Loose coarse to medium sand; medium compact fine sand	4
9	Compact sand-clay soils	3
10	Loose fine sand; medium compact sand-inorganic silt soils	2
11	Firm or stiff clay	1.5
12	Loose saturated sand-clay soils; medium soft clay	1

*Defined as maximum allowable presumptive bearing value.

pressure at the interface between steel H-piles and the surface of hard rock. It is true that the stress in the steel, at that point, greatly exceeds the allowable bearing value of the rock at that point, expressed in tons per square foot. However, the stress is rapidly dissipated from the surface of contact, so that there is no danger of failure by crushing. The ultimate strength of a small area—0.24 square feet for the largest H-pile—completely surrounded and confined by a great extent of bedrock is many times greater than the strength of a rock core in a testing machine. At most, there will be slight settlement, due to rock elasticity or grooving, well within acceptable limits for any foundation. Section 14 contains a number of test records of H-piles driven to rock, with stresses under test load as high as 35,000 psi. In every case, net settlement after rebound did not exceed a small fraction of an inch.

As a reference, Table 9B gives the "maximum allowable presumptive bearing capacities" for various grades of rocks and soils as stated in a large eastern city's current building code.

Friction piles

In actual practice, few piles are exclusively end-bearing. Most develop at least some of their bearing capacity from the soil. The converse is true in the case of friction piles, with a part of the total load being delivered as end-bearing at the tip. It is convenient, however, to consider the friction pile as distinct from the end bearing pile previously discussed.

Friction piles function as load transfer units, distributing the imposed load to the soil in which they are imbedded. The rate of transfer of the load, from the cap to the tip, is affected by many factors. Principal among these are the variations in the strength characteristics of the soils through which the piles must pass, and the variations in their ability to accept the load from the piles. The pick-up of load from the pile face by friction, bond, or shear is a closely associated interaction between pile and soil. An integral part of this interaction is the distribution of the load throughout the mass of the soil according to some pattern, along all or part of the length of the pile. So closely associated is the interaction, in fact, that the pile foundation has been described more accurately as a "pile-soil system."

Minimum spacing of friction piles

Spacing of friction piles is limited by the presumptive bearing capacity of the soils ultimately supporting the load. The pattern of distribution of load to these soils, and their condition of stress, is extremely complex and can be determined only by elaborate instrumentation not justified except for research. Also, results could not be applied to other piles on the same site. Reasonable and conservative methods of pile selection and group arrangement, conforming to local experience and combined with adequate load-testing, are the only present alternatives.

It is common practice to assume that where a friction pile is properly installed by seating in a good foundation soil, part of the load is carried by end-bearing at the tip. The balance is distributed by friction assumed to start at the elevation where the pile enters such material.

The following is a method of approximation quoted from *American Standard Building Code Requirements*, published by the American Standards Association, and sponsored by the American Society of Civil Engineers. It is illustrated in Figure 9C.

> "From the elevation at which a given load is considered to be transmitted to the soil, such load may be assumed to spread uniformly

within an angle of 60 degrees with the horizontal starting at a polygon circumscribing the group or row of piles being analyzed."

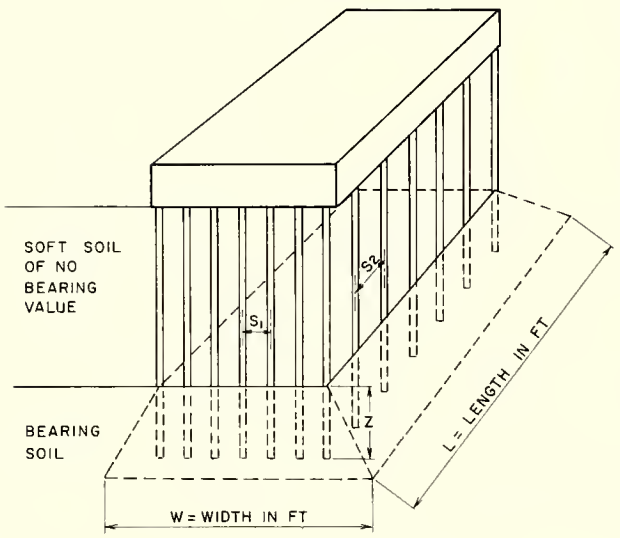
For any bearing stratum with the bearing capacity estimated or determined through load tests, the above approximation will yield a conservative minimum permissible spacing for friction piles.

A spacing of friction piles in a group should be selected so that the capacity of the group will equal the sum of the capacity of the individual piles. This can be determined by successive trials. A trial spacing of 2 ft 6 in. center-to-center may be used to check the total load of the group against the allowable bearing capacity of the soil at the tip, assuming the 60 degree distribution from elevation of penetration in bearing material as described above. If the unit pressure exceeds the allowable unit pressure, the pile spacing should be increased.

To provide for deviation in driving, the minimum spacing should be not less than 1.75 times the diagonal dimension of the pile, nor less than 2 ft 6 in.

Refer to Table 9B for presumptive bearing capacities.

FIGURE 9C



$nm\ R_a \leqq w \cdot L \cdot q_s$

where

n = number of piles per row.

m = number of rows.

R_a = allowable load per pile, in tons.

$w = (n-1)S_1 + 2Z$ cot 60 degrees.

$L = (m-1)S_2 + 2Z$ cot 60 degrees.

S_1 = center-to-center distance between piles across the width, in feet.

S_2 = center-to-center distance between piles along the length, in feet.

q_s = allowable soil bearing value, in tons per square foot.

Z = distance from bearing elevation to bottom of pile.

Bond with concrete

In most cases, bond between the H-pile and concrete is relatively unimportant; the top cross section of the pile will serve adequately to transmit the entire load to the concrete with safety. Refer to *Pile Caps*, in Section 11. Bond is important, however, where uplift may be a factor. Three basic conditions exist in which bond varies.

The first condition is where concrete is poured under dry conditions. In this case, it is possible to thoroughly clean the piles, removing all oil and dirt. This should be done just before pouring. A unit working stress of 40 psi is suggested.

The second condition is where tremie concrete is poured around piles that have been driven underwater into a relatively undisturbed bottom. A value of 20 psi should be used here, for the bond.

In cases where soil underwater has been excavated after the piles have been driven, the soil adhering to the steel will prevent a bond from being formed and no bond value should be allowed.

Some 40,000 tons of Bethlehem H-piles, in lengths from 50 ft to nearly 200 ft, were used along the Connecticut Turnpike.

Section 10

INCREASING CAPACITY WITH LAGGING

Occasionally, a soil condition is found that requires attachments to H-piles to increase their bearing capacity. Such a condition existed at the site of a pier constructed in Venezuela several years ago. The soil was originally an alluvial deposit of a peculiar type of micaceous sand from which some constituent material had been dissolved by ground water. The residual fraction was in a loose state and could not develop high frictional resistance on a steel face.

A plain H-pile was driven 40 feet into this very unusual soil and load tested. The result was disappointing; the failure load was 93 tons, while the desired design load was 90 tons per pile.

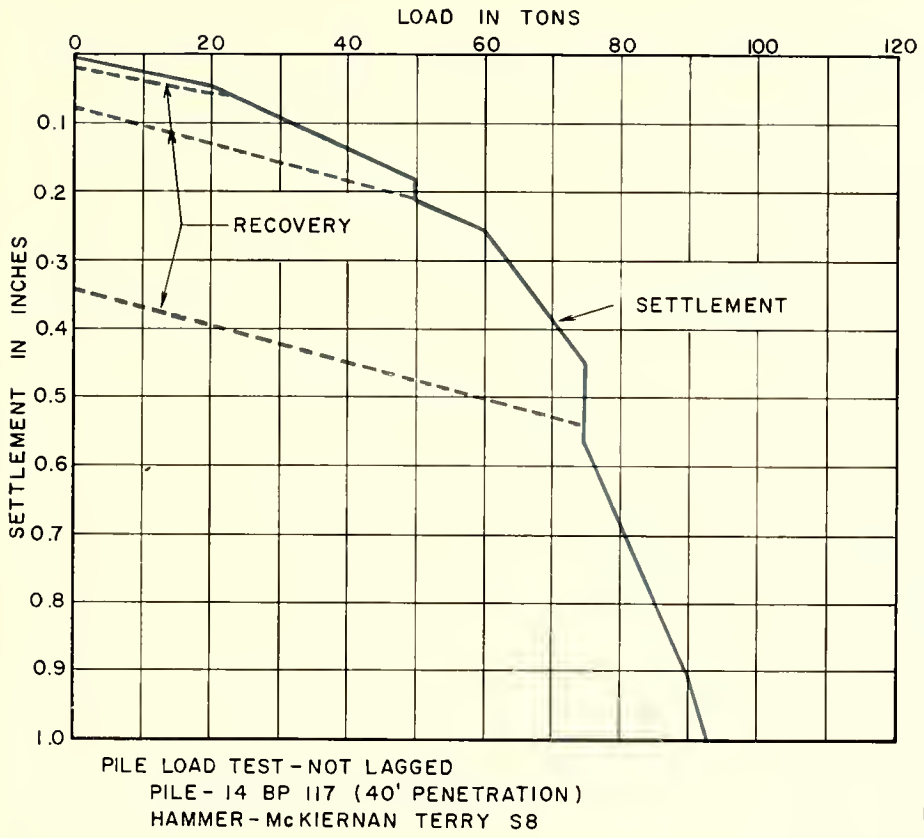
A very satisfactory solution was obtained by equipping the piles with timber lagging, as shown in Figure 10A. The lagging was only 12 feet long, and was reinforced top and bottom with steel. The shear was taken by the steel; the function of the timber was to displace the soil and build up a bulb of pressure.

The load-settlement curve, Figure 10B, shows that the lagged pile tested to 200 tons with practically zero net settlement. The depth of penetration was only two feet greater than for the plain pile.

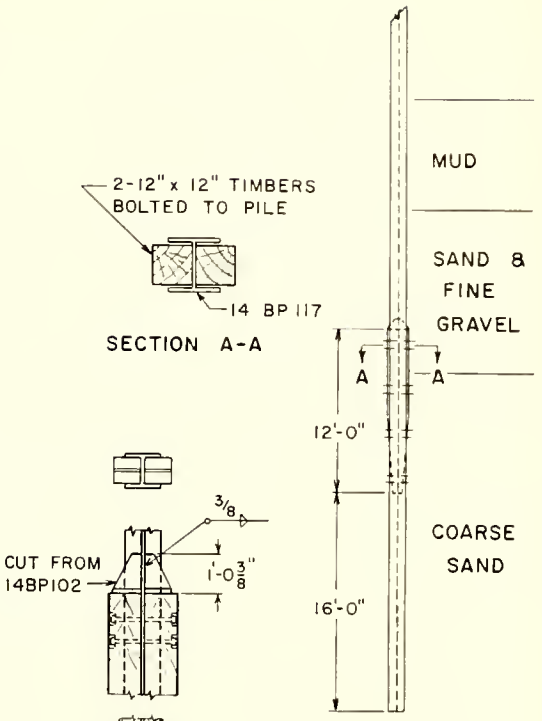
Experiment demonstrated that taper in the lagging was decidedly better than parallel-sided lagging. An advantage is that the taper can be obtained in a comparatively short distance, and the percentage of taper is therefore greater than for a long tapered pile. There is evidence to show that in certain soils, bearing capacity increases rapidly with the degree of taper. See Highway Research Board Special Report 36—*A Study of the Comparative Behavior of Friction Piles.*

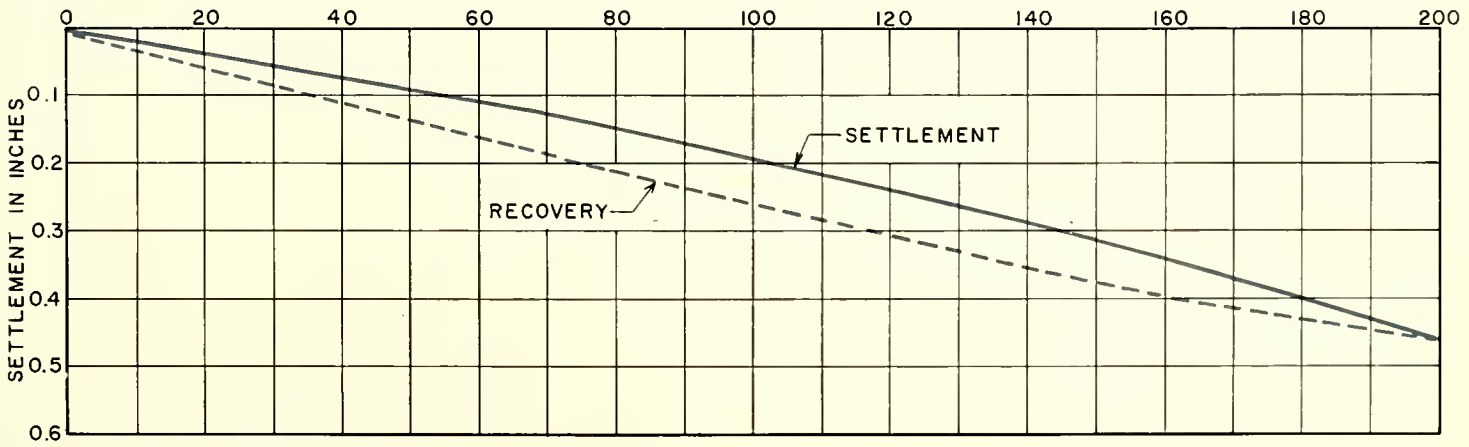
FIGURE 10A—12-ft long tapered timber lagging, reinforced top and bottom with steel to take shear stresses, for a pier in Venezuela.

FIGURE 10B— H-PILE TEST, LOADING PIER—PUERTO DE HIERRO, VENEZUELA



PILE LOAD TEST - NOT LAGGED
PILE - 14 BP 117 (40' PENETRATION)
HAMMER - McKIERNAN TERRY S8





PILE LOAD TEST - LAGGED
PILE - 14 BP 117 (44' PENETRATION) - 113 B.P.F.
HAMMER - McKIERNAN TERRY S8

Section

11

ACCESSORIES

Pile caps

Many engineers specify steel plates over the piles, to distribute the load from the top of H-piles into the concrete caps, footings, or foundations. Table 11A gives the size of plates required to develop the designated loads for the various sizes of piles.

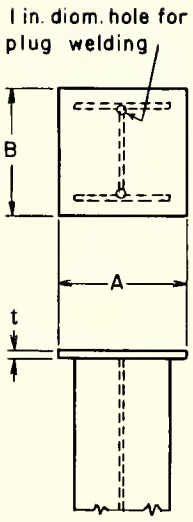


TABLE 11A—PILE CAPS

Load in Tons	*BP 8x36*				*BP 10x42 BP 10x57*				*BP 12x53 BP 12x74*				*BP 14x73 BP 14x89 BP 14x102 BP 14x117*			
	A	*B*	*t*	*Wt, lb*	*A*	*B*	*t*	*Wt, lb*	*A*	*B*	*t*	*Wt, lb*	*A*	*B*	*t*	*Wt, lb*
20	8	8	5/8	11												
25	8	9	5/8	13												
30	9	9	5/8	14												
35	10	10	5/8	18	10	10	5/8	18								
40	10	11	5/8	19	10	11	5/8	19	12	12	5/8	26				
45	11	11	3/4	26	11	11	5/8	21	12	12	5/8	26				
50	12	12	7/8	36	12	12	5/8	26	12	12	5/8	26				
55	12	13	7/8	39	12	13	5/8	28	12	13	5/8	28				
60	13	13	1	48	13	13	3/4	36	13	13	5/8	30				
65	13	14	1	52	13	14	3/4	39	13	14	5/8	32	14	14	3/4	42
70					14	14	7/8	49	14	14	3/4	42	14	14	3/4	42
75					14	15	7/8	52	14	15	3/4	45	14	15	3/4	45
80					15	15	1	64	15	15	7/8	56	15	15	7/8	56
85					15	16	1	68	15	16	7/8	59	15	16	7/8	59
90					15	16	1	68	15	16	7/8	59	15	16	7/8	59
95					16	16	1 1/4	90	16	16	1	72	16	16	7/8	63
100					16	17	1 1/4	97	16	17	1	77	16	17	7/8	68
110									17	18	1 1/8	98	17	18	7/8	76
120									18	18	1 1/4	115	18	18	1	92
130									19	19	1 3/8	140	19	19	1 1/8	115
140													19	20	1 1/8	121
150													20	20	1 1/4	142
160													21	21	1 3/8	172
170													21	22	1 3/8	180
180													22	22	1 1/2	205
190													22	23	1 1/2	215
200													23	24	1 5/8	254

These caps are based on an allowable bearing value of concrete not exceeding 750 psi.

Transmittal of load to H-piles without steel pile caps

An adequately designed reinforced concrete pile cap does not require steel plates to transmit the load to the H-piles. Research by the State of Ohio Department of Highways in 1947 demonstrated this with a very comprehensive series of tests, the

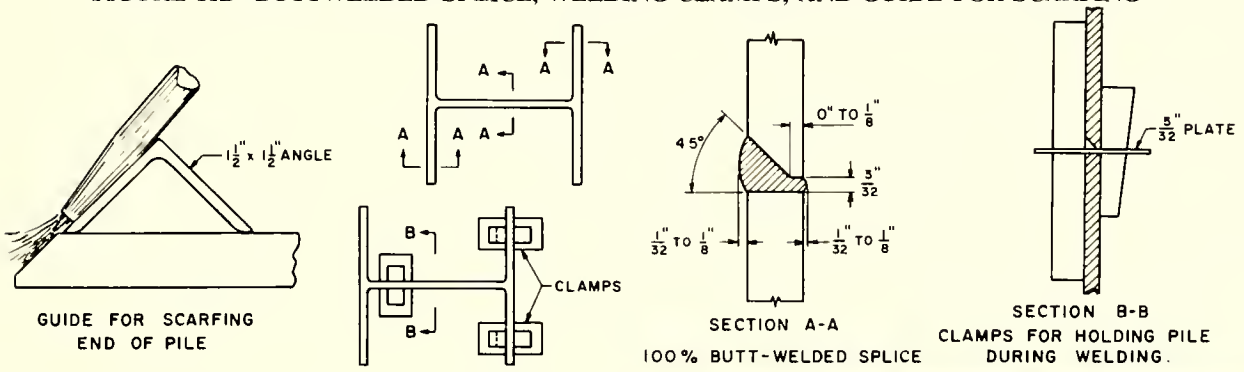
results of which were published in their Research Report No. 1, *Investigation of the Strength of the Connection Between a Concrete Cap and the Embedded End of a Steel H-pile.* The following is quoted from their conclusions: ". . . the evidence is believed conclusive that where the top of a steel H-pile is embedded in a concrete cap or footing, if the pile itself is of adequate section and the concrete member is of adequate size and arrangement and properly reinforced for the pile reactions, there need be no concern regarding the strength of the connection for compressive force, and it is unnecessary to provide a bearing plate or other auxiliary device at the top of the pile."

Splices

H-pile splices should be designed to develop the full strength of the pile both in bearing and bending. This can be done most economically with buttwelded splices. Some engineers prefer plate splices, with the strength developed by welding between the edges of the plates and the H-pile; others go to the extreme of using plates and buttwelding combined, but this seems a needless expense. A 100 per cent buttwelded splice is recommended. It is customary to order piles that are to be spliced, cut to mill tolerances. Milled ends are unnecessary. Field scarfing preparatory to butt-welding is done by torch-cutting.

The buttwelded splice shown in Figure 11B was first shown in our H-pile catalog published in 1948. Since then it has been used extensively with uniformly successful results. For example, 70,000 tons of H-piles 140 feet long were driven at Bethlehem's Sparrows Point plant in 1955-56 using this splice. All piles were driven successfully, without a single splice failure.

FIGURE 11B—BUTTWELDED SPLICE, WELDING CLAMPS, AND GUIDE FOR SCARFING



Splicing is often performed before the piles are placed in the leads to be driven, in order not to delay operation of the pile-driving equipment. One of the important advantages of H-piles is that they can be handled without damage in very long lengths—sometimes in excess of 200 feet. Figure 11C shows a buttweld splice being completed on a pile lying horizontally on skids. This method requires that the pile be turned over several times during the welding operation. Where a large number of piles must be spliced, the roller jigs shown in Figure 11D are an advantage, since the piles can be rotated easily to permit downhand welding.

FIGURE 11C—The buttweld splice, easily made with downhand welding procedures, is entirely suitable for most driving conditions.

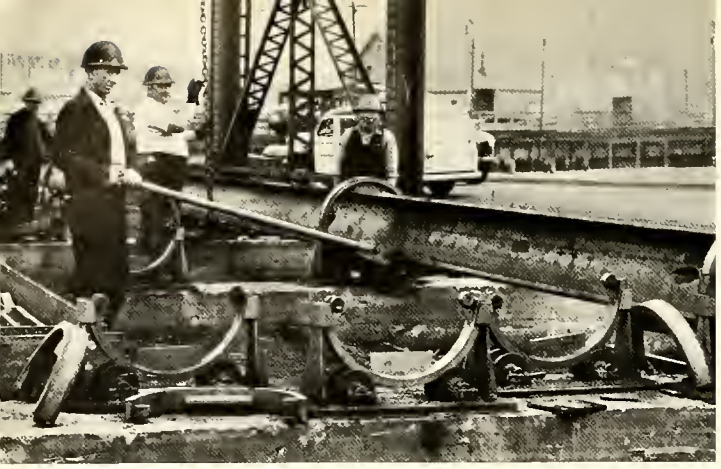


FIGURE 11D
For the John Hancock Building, built in Boston in 1946, H-piles were welded in these special roller jigs that permit all downhand welding.

Pile point reinforcement

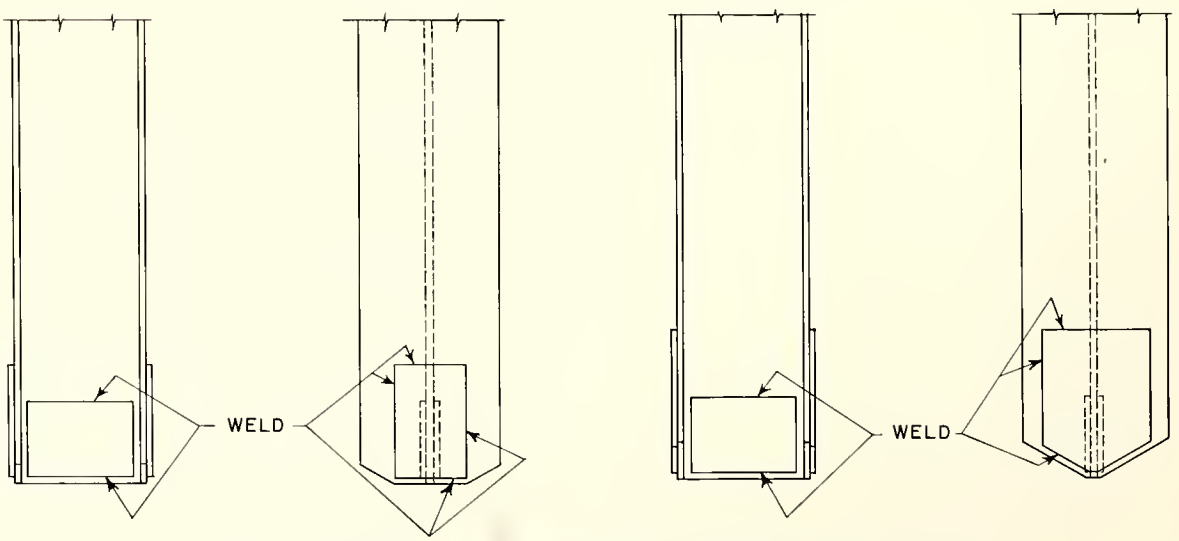
Pile point reinforcement is seldom needed. Conditions for which pile points might be considered are:

1. Where piles must penetrate a layer of soft rock or other hard material that is underlain by soft soil which also must be penetrated before the piles reach the final bearing stratum. For instance, at Bethlehem's Sparrows Point plant, during the construction of a pier, it was necessary for piles 160 feet long to penetrate a layer of solidified slag at a depth of 20 feet. Pile points made it possible to punch through the slag. Only two per cent of the piles required points.
2. Where piles are driven to sloping hard rock, reinforcement such as shown in Figure 11E will reduce distortion of the bottom of the pile.

Conditions for which pile points should not be considered include the following:

1. For friction piles, as enlargement of the bottom of the pile will greatly reduce the skin friction on the surface of the pile above the point.
2. Where the bearing stratum is hardpan or soft rock that can be penetrated for several feet. The enlarged bottom of the pile will cause the pile to refuse at a higher elevation, and the pile will not develop the required combination of point bearing and skin friction as efficiently as a pile without point reinforcement. See *End-Bearing Piles*, Section 3.

FIGURE 11E—TYPES OF POINT REINFORCEMENT FOR H-PILES



Web plates welded top and bottom edges only; flange plates welded all around.

Section 12

DRIVING H-PILES

Pile hammers

H-piles are most commonly driven with single- or double-acting hammers operated with steam or compressed air, although the use of diesels and other types is growing. The single-acting hammer derives its energy from the fall of a ram that is raised inside the hammer casing by steam (or air) pressure. In the double-acting hammer, the ram is also raised by steam (or air) pressure, but at the top of the stroke the pressure is valved to the top of the ram, adding to the energy of the free fall.

A double-acting hammer is generally operated at more blows per minute than a single-acting hammer of equal energy per blow. The former may be preferred where piles must be driven through a considerable depth of soil where the penetration per blow is small; the greater frequency of the blows will give faster penetration. The single-acting hammer might be used where the soil above the bearing stratum can be penetrated rapidly—an "easy driving" condition. Both types of hammer must be operated at the rate specified by the manufacturer, to develop the published energy per blow. The greater number of blows per minute specified for the double acting hammer makes necessary steam boilers or air compressors of greater capacity.

In recent years, diesel hammers have come into use. Their principal advantages lie in the elimination of boilers or compressors, and in their greater mobility. A disadvantage of some types lies in frequent stalling under easy driving condi-

Here, an H-pile is used as a guide for driving another H-pile; difficult site conditions have precluded the use of leads.

tions, requiring frequent restarts. Also, rating the energy per blow presents difficulties.

Drop hammers are occasionally used for driving a limited number of piles in an isolated location where the amount of construction equipment must be held to a minimum and it is desirable to eliminate the cost of installing and operating a steam boiler.

The use of vibrating devices for driving piles is a recent development. In certain soils the speed with which piles have been driven and pulled is phenomenal. However, the method is not applicable for all soil conditions. At present the amount of published information is limited, although the manufacturers are acquiring a backlog of experience. One difficulty is the present lack of a reliable method of estimating the bearing capacity of piles installed by this method.

Section 18 contains a table of specifications for the hammers most commonly used today.

Driving piles under water

A very common procedure in the construction of bridge piers is to drive piles under water, sometimes to considerable depths. A depth of 40 to 50 feet is not uncommon. Some hammers are especially equipped for this service.

Size of hammer

Selection of the hammer size is influenced by various factors such as the weight of the pile, design load per pile, pile length, whether the capacity of the pile is to be determined by formula or established by load tests, and whether the driving will be hard or easy until the bearing stratum is reached. No hard and fast rule can be given. Obviously, a heavier hammer is required for a pile designed to carry heavy loads than for one that will carry a light load, and for a long BP 14 x 117 than for a short BP 10 x 42. For relatively hard driving to great depths, a heavy hammer will be more efficient than a light hammer. If the capacity of the pile is determined by formula, the size of hammer must be selected for the formula specified. For instance, for a 50-ton pile, the lightest hammer that should be used with the *Engineering News* formula discussed on page 50 is one with an energy per blow of 10,000 ft-lb, assuming that the pile is driven to a resistance of 0.1 in. per blow. For the same pile, a 20,000 ft-lb hammer would be required where the modified *Engineering News* formula is specified. This points up another weakness of pile-driving formulas. At Bethlehem's Sparrows Point plant a test pile 140 feet long driven with a 15,000 ft-lb hammer failed at 385 tons, which led to a design load of 150 tons per pile. The pile was driven to a resistance of 0.8 in. per blow. The *Engineering News* formula would have required a 27,000 ft-lb hammer, and the modified *Engineering News* formula would have required a 57,000 ft-lb hammer.

Driving caps

A most important requirement in driving H-piles is maintaining the hammer coaxial with the pile. A hammer suspended from a slack line may quickly buckle the flanges and require the pile to be trimmed with a torch before driving can proceed. Another important requirement is that the hammer be equipped with a driving cap with grooves in the base that loosely conform to the shape of the pile. Examples of driving caps are shown in Figures 12A and 12B. The cap should be loosely attached to the hammer so that it will at all times rest squarely over the entire surface of the pile. The bearing surface of the grooves must be true, without roughness. Any high spot can cause the flanges or the web to start buckling. Once

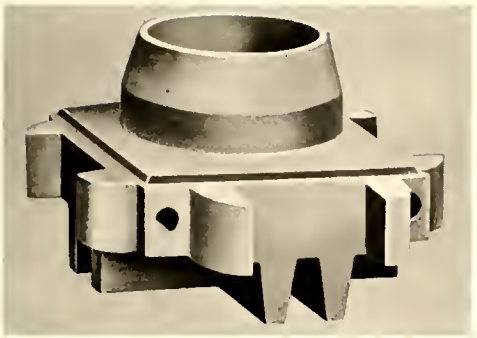


FIGURE 12A

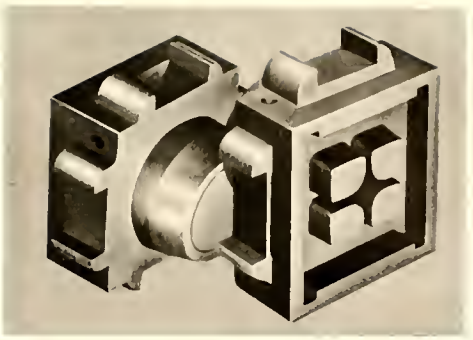


FIGURE 12B

Two types of driving caps are illustrated above.

started, buckling progresses rapidly, so that the pile becomes wedged in the cap grooves and requires torch-cutting to free it. This burning roughens the grooves still more, and aggravates the cause of buckling. H-piles will withstand extremely hard and prolonged driving when proper methods and equipment are used.

Driving to refusal on rock

Where H-piles are driven to refusal on hard rock, care should be taken to guard against unnecessary damage to the bottoms of the piles. It is not good practice to continue driving after the piles are firmly seated on the rock. The following definition of refusal is suggested.

Specification for Refusal:
A pile should be considered driven to refusal when 5 blows of an adequate hammer are required to produce a total penetration of one quarter-inch. Driving should then cease. Where borings indicate that the pile tip has not reached rock or penetrated into bearing stratum, the nature of a probable obstruction should be investigated and the method of removal or bypass be decided by the engineer-in-charge.

Driving methods

The proper method of driving H-piles in any particular case depends upon many related factors: the nature of the structure, size and number of piles to be driven, the required driving accuracy, physical characteristics of the site, whether a dry land or a water job, and many others.

A simple setup consisting of a diesel crawler crane, swing leads, and a drop hammer is shown in Figure 12C, at left. This equipment is suitable for driving a limited number of isolated piles where the amount of construction equipment must

FIGURE 12C—Rig for driving H-piles with drop hammer.

FIGURE 12D
Two rigs suitable for large jobs.

FIGURE 12E

be held to a minimum. A driving cap must be used with a drop hammer to hold the pile centered in the leads and to protect the top of the pile.

Driving rigs more suitable for a large job are shown in Figures 12D and 12E, left. Stakes are driven in the ground to locate the piles. This method is applicable where the chief concern is to drive as many piles per day as possible, and where a reasonably liberal tolerance is permitted in plumbness and pile position. By this method the final position of the top of the pile should be within about three in. of its theoretical location and it should be plumb within about one in. in five ft of length. Figure 12F shows a large group of piles, on the same job shown in Figure 12G, but after the piles were cut off and capped. Figure 12F gives a good idea of the alignment obtainable by this method.

An A-frame pile driver of the type shown in Figure 12H often is used to drive H-piles. An advantage of this rig is that the pile can be held in position by guides at the bottom of the leads and by a driving cap at the top, the cap being fitted to slide in the leads. Thus, the pile can be held so that it cannot twist around a vertical axis. Also, it is held plumb and in position with greater precision than is possible with swinging leads.

However, this rig is more cumbersome and less mobile than those shown in Figures 12F and 12G, since the entire rig must be shifted and spotted for each pile.

In cases where there is a tendency for piles to rotate, and the rotation is considered objectionable, it can be prevented by means of a guide frame like the one shown in Figure 12J, or by using a driving cap.

Occasionally, H-piles are required to be driven with very close tolerances. This requirement sometimes is dictated by the consideration of appearance, or by the necessity of field fabrication of bracing members. Figure 12L illustrates both of these requirements. In order to obtain this degree of precision, it is necessary to erect an accurate guide frame at or close to the ground or water level, with a second frame about 20 to 30 ft or more above it,

FIGURE 12F—H-pile foundation, with all piles cut off and capped. Note alignment of piles.

FIGURE 12G—H-piles for an industrial foundation, as driven.

FIGURE 12H — A heavy-duty A-frame type of driving rig.

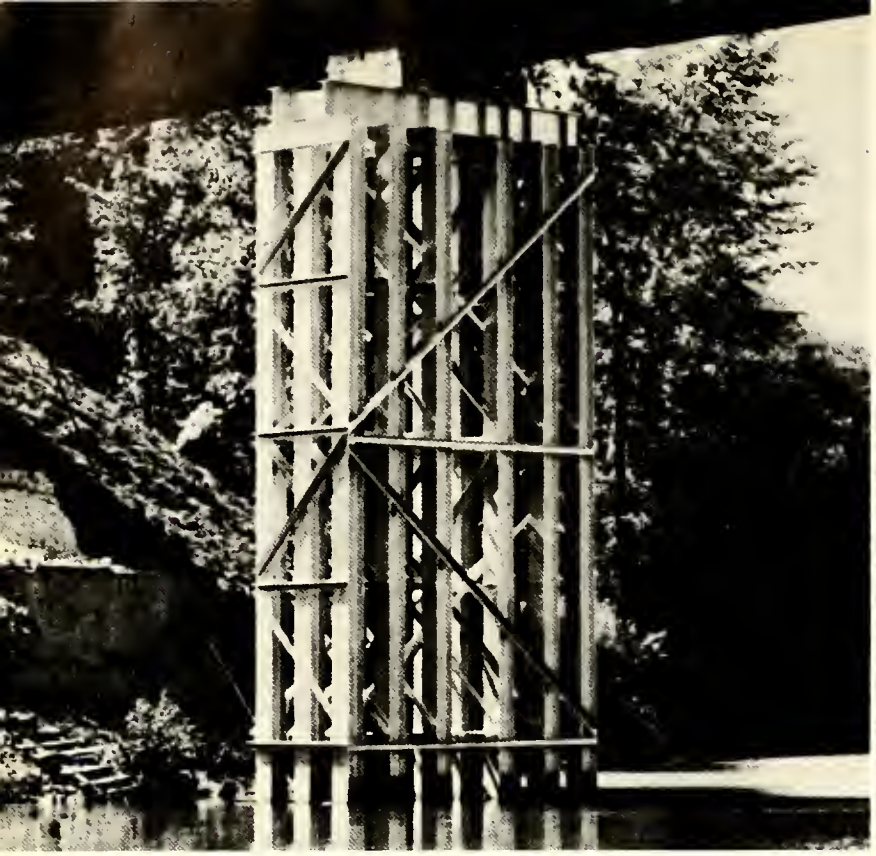


FIGURE 12L—An example of precise driving of extremely long H-piles.

FIGURE 12M—A rig for driving batter piles.

FIGURE 12J—Guide frame for driving H-piles.

FIGURE 12K—Batter piles set in guide frames, driven to the first stage. H-piles were driven to this stage by a hammer with swinging leads.

depending upon the length of the piles and the distance they extend above ground or water level.

In this manner, the piles are held plumb and exactly in position. Driving accuracy within ¼ in. of exact location is possible by this method. Where a double frame is used, the piles may be set up in a group beforehand and driven by means of a hammer suspended from a crane or derrick boom. The hammer may be held and guided by swinging leads that extend part way down the pile.

H-piles may be driven on a batter by means of a pile driver equipped with rotatable leads that can be inclined to the

FIGURE 12P—A view of H-piles being driven by a rig mounted on the stern of a scow. For an outfitting pier in Hoboken, N.J., these piles were of various lengths up to 210 feet.

FIGURE 12Q—Another view of the rig shown in Figure 12P. Note the driving skirt, which held the hammer centered on the pile.

FIGURE 12R—Driving 45-degree batter piles from a scow-mounted A-frame.

desired angle. Or, a rig may be equipped with swinging leads, as in Figure 12M, which can be rotated to any prescribed angle or combination of angles. Another method is to set the piles in a rigid frame as shown in Figure 12K. The piles shown are foundation piles for a bridge pier. The first stage of driving was done with a steam hammer operating in swinging leads; the second stage carried the tops of the piles 20 ft below water.

An interesting example of guiding H-piles during driving is shown in Figure 12P. Here the rig was mounted on the stern of a derrick scow. Figure 12Q is a general view of the rig in operation.

Figure 12R shows an ingenious method of driving 45-degree batter piles to act as anchors for a sheet pile bulkhead. The short inclined leads can be raised and lowered in the main vertical leads. After the hammer reaches the lower end of the short leads, it is raised to the top, and the rig moves forward until the hammer is again centered on the pile and in position to resume driving. This rig is economical for driving a limited number of batter piles, since an ordinary floating pile driver can be converted in this manner at small cost.

Section 13

PILE-DRIVING FORMULAS

The use of pile-driving formulas for determining the safe load for which piles can be designed is a practice long established and widespread. Practically every building code includes at least one pile-driving formula. There is no doubt that these formulas are misused in some cases, and that their use should be regulated by certain restrictions, because they are not universally applicable.

Until recently, the published data on pile testing was insufficient and inadequate to attempt any study correlating computed safe or ultimate loads from formulas, and actual capacities. The large majority of pile tests were carried only to the point of determining whether a pile would safely support a predetermined load, and were rarely extended to ultimate failure. The applicability of formulas was thus left, at least partially, in doubt. In 1958, the Highway Research Board published Special Report 36, *A Study of the Comparative Behavior of Friction Piles*, and in 1961 Special Report 67, *Records of Load Tests on Friction Piles*. These reports cover load tests of more than 600 piles of many types, including H-piles, cast-in-place concrete piles, pipe piles, concrete poured into drilled holes, and others. Many of the tests were carried to failure, and it is now possible to test the reliability of the various pile-driving formulas.

The formula in widest use is the *Engineering News* formula. If the reliability of this formula is investigated by comparing its so-called safe load with the ultimate load determined by test as reported in the above publications, it will be found that the results are very erratic. The ratio of the "safe load" to the ultimate load by test is in some cases less than unity, and in others is unnecessarily high for good design, judged on the twin requirements of safety and economy. This applies to all types of piles and to most types of soil, both granular non-cohesive soils and clay.

A modification of the *Engineering News* formula that has received considerable acceptance decidedly increases the safety factor where the original formula gives unsafe results, but it increases still further the safety factors that were already unnecessarily high.

The two formulas mentioned are the following:

Engineering News . . . . . $R_a = \frac{2E}{S + 0.1}$

Modified *Engineering News* . . . . . $R_a = \frac{2E}{S + 0.3}$

where
R_a = safe load, lb
E = energy per blow, ft-lb
S = final penetration per blow, in.

In addition to the above formulas, there are a number of so-called dynamic formulas so complicated they are not practical for general use. Furthermore, their complexity does not add to their accuracy, since the results they give are in many cases equally erratic.

Pile load test preferable to formula

Generally speaking, where the size and nature of the project justifies the cost, loaded test piles are far more reliable than formulas for determining the safe design load per pile. Engineers prefer to keep the load per pile low where formulas must be depended upon, due to the uncertainties inherent in their use. Substantial economies can be effected by utilizing the available strength in steel piles, once the factors of ignorance can be eliminated by reliable information, based on properly conducted tests.

It is suggested that the use of pile-driving formulas be restricted to small projects where the cost of load tests would be too high a percentage of the total cost of the piles.

LOAD TESTS OF H-PILES

It is common practice for the engineer or the local building code to require tests to prove that a specified pile can carry the design load with an acceptable factor of safety. In Section 13—*Pile Driving Formulas*—it is pointed out that a formula may indicate a "safe" load that actually has very little or no factor of safety. On the other hand using a formula may result in an "allowable" load so low in comparison with the capacity of the piles being used that more piles are purchased and installed than would be necessary to provide a good foundation.

Many building codes and specifications take into account the unreliability of pile-driving formulas. The New York Building Code, for instance, limits the use of a formula to a maximum of 30 tons for friction piles and 40 tons for piles driven to rock. The standard specifications for highway bridges of the American Association of State Highway Officials specifies very low loads per pile when the allowable load is determined by formula, and greatly increased loads where the design load is substantiated by load tests.

Where H-piles are driven to solid rock, load tests may often be omitted with good reason. This is particularly true in cities where the character of the rock is well established, and the data from previous tests in the vicinity are available.

The following procedure for load testing is taken from the *Tentative Method of Test for Load-Settlement Relationship for Individual Piles Under Vertical Axial Load*, ASTM specification D1143-61T:

"The head of the pile shall be cut off level or shall be capped in such a manner as to produce a horizontal plane bearing surface. A steel plate shall be set on top of the pile. . . .

"The total test load shall be twice the anticipated working load on the pile and shall be applied in increments amounting to 25, 50, 75, 100, 125, 150, 175 and 200 per cent of the anticipated working load. Settlement readings made to an accuracy of 0.01 in. shall be taken before and after the application of each new load increment and at 2, 4, 8, 15, 30, and 60 min, and every 2 hr until application of the

next load increment. Additional load shall not be applied until the rate of settlement under the previous increment is less than 0.01 in. in 1 hr or until 2 hr have elapsed, whichever occurs first. When the loading has been completed, the full test load shall remain on the pile for 24 hr or for a longer period if the necessity therefor is indicated by the rate of settlement of the pile, and settlement readings shall be taken at 6-hr intervals during the period and at the end of that period. As an alternative method of loading, the specified load increments may be added in constant time intervals of not less than 30 min and preferably of 1 hr. Settlement readings shall be made immediately before and after the addition of each load increment and at not less than three specified times between load increments.

"During the unloading of the pile the rebound shall be measured when the load remaining on the pile amounts to 75, 50, 25, 10, and 0 per cent of the full test load. . . .

"The report of the load test shall include the following information:

(1) A description of soil conditions at the location of the test pile,
(2) A description of the pile and its driving record, including the number of hammer blows per foot throughout the pile length and the final driving resistance in blows per inch for the last 3 in. of driving,
(3) A description of the hammer and its actual rate of operation during the driving of the test pile,
(4) A tabulation of the loads and settlement readings during the loading and unloading of the pile,
(5) A graphic representation of the test results in the form of a time-load-settlement curve, and
(6) Remarks concerning any unusual occurrence during the driving or loading of the pile."

In addition to the above it is suggested that the final reading of net settlement be taken 24 hours after unloading is completed, or longer if it is obvious that the pile is continuing to slowly recover. Long piles, in clay soils particularly, may require considerable time to fully recover.

Methods of testing

There are two methods of testing in general use. One is the anchor-pile method, and the other is the platform method, where a loaded platform provides the test load.

ANCHOR-PILE METHOD

In this method, three piles are driven in a row; the center pile is the test pile, and the outside piles furnish the reaction. It is used where adequate resistance to uplift is developed in the reaction piles. As a rule the three piles used in the test can remain as part of the permanent structure. This method is particularly suitable:

(1) where the available space is restricted, as in a cofferdam;
(2) where the test must be conducted in open water;
(3) where a number of tests must be made in different locations;
(4) where the material required for the dead load for a platform test is not readily available.

The method is illustrated in Figure 14A. Two I-beams were attached to the reaction piles, forming a yoke. Two hydraulic jacks, positioned between the yoke and the test pile, supplied and measured the test load. Settlement readings were obtained by means of a dial gage fastened to an independent rigid support, with the plunger of the gage bearing against an attachment on the test pile. It is customary to check the gage readings by attaching a scale to the test pile and to read the settlements with a surveyor's level which is referenced to an independent bench mark.

FIGURE 14A—Anchor-pile method of load testing H-piles.

LOADED-PLATFORM METHOD

Figure 14B shows a platform supported on four piles and loaded with steel ingots. The load was applied and measured by a hydraulic jack pushing upward against the platform, with part of the load being supplied by the uplift resistance of the four piles which support the platform. The settlement was measured with dial gages to the nearest 0.001 inch. This method is suitable for locations where the surface ground is too soft to support the entire test load.

On many jobs the H-piles for the foundation can be used to advantage for the test load. They are placed in layers on timber cribs, the axis of the piles being alternated 90 degrees between layers. The weight of the load should exceed the test load by about 15 per cent, so that the H-piles are never actually lifted. Settlement measurements are usually made with dial gages, and checked with a surveyor's level.

FIGURE 14B—Loading test of H-piles driven to shale. The testing bridle shown is held down by means of four steel anchor piles and 130 tons of ingots.

Section

15

INTERPRETATION OF LOAD TESTS

There are numerous rules for the interpretation of load tests. Most of these appear in the various codes of cities and states and other regulatory bodies. They vary a great deal. Some of them specify a maximum net settlement under the test load, others a maximum gross settlement, while still others undertake to determine the yield point of the pile from the shape of the load-settlement curve.

Figure 15A gives the load-settlement curve of a test pile driven at the Bethlehem Steel plant at Lackawanna, New York. As a rule, if the test load is removed with sufficient time between decrements, the recovery line is straight. However, in clay soils full recovery for each decrement takes too much time, and for this reason

FIGURE 15A

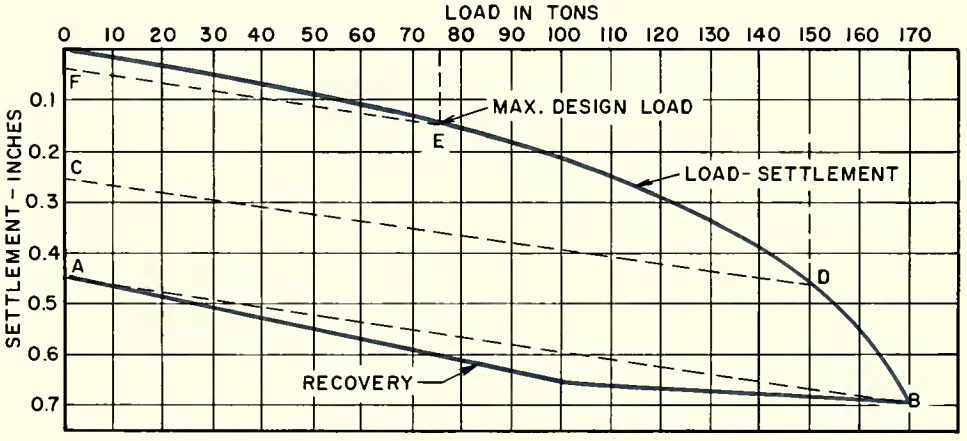
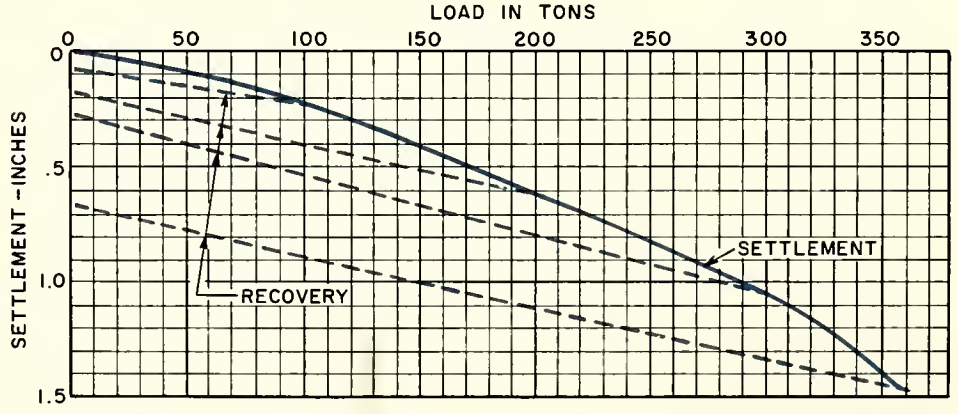


FIGURE 15B



it is customary to hold each load approximately one hour during the unloading cycle. After all of the load is removed, the final settlement reading is taken after all upward movement of the top of the pile has ceased. Point *A* in Figure 15A records this final net settlement.

When the test load is applied in increments, with the load being removed after each increment, the recovery lines are approximately parallel, provided that adequate time is allowed for full recovery. Figure 15B shows an example of a load-settlement curve for a test pile at Bethlehem's Sparrows Point plant, indicating recovery lines for increment loading and removal.

A suggested method for determining maximum permissible design load from the load-settlement curve would be one of the following:

1. Where the net settlement after removing the test load does not exceed ¼ in., the design load may equal one-half the test load, subject to other applicable limitations.

2. Where the net settlement after removal of the test load exceeds ¼ in., proceed as follows:

   Refer to Figure 15A. Connect points *A* and *B* with a straight line. Using the characteristics of recovery lines mentioned above, draw *CD* parallel to *AB*, point *C* being selected to correspond to a net settlement of ¼ in. The maximum design load (point *E*) should be one-half the load intercepted by point *D*. The line *EF* drawn parallel to *AB* is the approximate recovery line for the design load, and will invariably indicate very little net settlement.

There are building codes that permit a net settlement of 1/100 in. per ton of test load. This regulation is unrealistic, particularly for heavily loaded piles. For instance, for a 100-ton pile, with a test load of 200 tons, a net settlement of two in. would be acceptable. This amount of net settlement can never be developed unless a pile is loaded to failure, and permitted to settle continuously under the failure load.

Other codes stipulate a maximum gross settlement under test load of one in. This regulation does not take into consideration that the elastic shortening of an extremely long pile, made of an elastic material, may exceed the stipulated limit, but not be overstressed. For instance, if a purely end-bearing steel pile 150 ft long is test-loaded to 24,000 psi—twice the design load—the elastic shortening is 1.5 in. if no load is transmitted to the soil overlying the bearing stratum. In practice, however, the gross settlement limitation rarely becomes restrictive, since much of the load is transmitted to the soil above the bearing stratum.

Section

16

GENERAL SPECIFICATIONS FOR H-PILES

1. Description. Steel H-piles shall be Bethlehem Sections of the size and weights shown on the plans. They shall be furnished, driven, and cut off in accordance with the plans and specifications.

2. Material. The material for H-piles (splices), (point reinforcement), (steel pile caps) shall conform to the Standard Specifications of the American Society for Testing and Materials for Steel for Bridges and Buildings, Serial Designation A7, or A36.

3. Lengths to be ordered. The lengths of piles on the plans are estimated and may be varied to conform to conditions actually encountered. The Engineer will approve the lengths of piles to be ordered, basing his decision upon the best

Over 9,000 tons of Bethlehem H-piles were used for the foundation of the Customs Court and Federal Office Building at Foley Square, New York City.

information available at the time the order is placed. When the Engineer deems it necessary, he may require the contractor to drive test piles in such locations and of such lengths as he may direct. The data obtained in the tests will be used by the Engineer in determining the lengths of piles to be used in the structure.

4. Splices. Splices shall be made by an approved procedure in accordance with the plans. They shall be so made and installed as to insure good alignment of the spliced parts. [*Splices should be designed to develop the full section of the piles and should preferably be butt-welded. See suggested details on page 41.*]

5. Bearing value. Each H-pile shall be driven to develop a bearing value of ____ net tons.

The Engineer may direct the contractor to make load tests at locations he will designate, to determine the lengths required to develop the specified bearing value. The test load may be applied by jacking against anchor piles; by jacking against a loaded platform; by loading a platform which is carried directly by the test pile; or by any other approved method. The proposed method shall be submitted to the Engineer for approval.

The load-test piles shall be of the same size as those specified and of lengths designated by the Engineer. The equipment and method used in driving the load-test piles shall be similar to those to be used in driving the piles for the permanent structure and shall be approved by the Engineer. Complete driving data shall be recorded, including the number of blows for each foot of penetration, and the final penetration for the last five blows of the hammer.

The test load shall be twice the specified bearing value of the piles. It shall be applied in equal increments, each not more than one-sixth of the total test load. After each increment has been applied, the settlement of the pile shall be recorded to the nearest 0.01 in. Each increment of load shall remain in place for at least two hours before the next increment is applied. The total test load shall remain in place at least 24 hours after settlement has ceased, at which time the total settlement shall be recorded. The load shall then be removed in decrements not exceeding one-fourth of the total test load, with intervals of not less than one hour, the net settlement after rebound being recorded after each decrement. Final net settlement shall be recorded 24 hours after the entire test load has been removed.

The test load pile shall be considered to have a safe-load value equal to the specified bearing value, if the net settlement after rebound, for the total test load specified above does not exceed ¼ in.

From the results of the load tests, the Engineer will determine the lengths of the piles, and the final penetration per blow required to develop the specified bearing value.

If load tests are not required, the bearing value of each pile in the structure shall be determined from the following formula. [*Give formula.*]

6. Driving. H-piles shall be driven by an approved hammer developing not less than ____ft-lb of energy per blow. [*See Section 12 for a discussion of driving; Section 18 for hammer ratings.*] In order to insure that this amount of energy is developed, the hammer shall be operated strictly in accordance with the manufacturer's recommendations, particularly for the final few feet of penetration of the pile.

Following are suggested specifications applicable to the conditions indicated in italics.

(*a*) *For a very large foundation with the piles fully embedded in the ground, where exact location is not of extreme importance, but where it is desirable to reduce driving costs to a minimum due to the large number of piles to be driven.*

The H-piles shall be driven in the locations shown on the plans. Deviation

of the location of the top of a pile from that shown on the plans shall not exceed three in. Where the deviation exceeds three in., and a redesign indicates a load on any pile exceeding 110 per cent of the design load, an additional pile or piles shall be driven. Piles shall be driven with a maximum permissible deviation of two inches in 10 ft from the vertical, or from the batter indicated on the plans. The hammer and piles shall be supported in rigid leads that extend to within two ft of the elevation at which the pile enters the ground. Before driving starts, the leads and pile shall be carefully plumbed. A suitable driving cap shall be provided to prevent undue damage to the top of the pile, and to hold the pile centered under the hammer.

(*b*) *For small individual foundations, such as column foundations for a building, for small bridge piers, or other foundations where fairly precise location of the pile is required.*

The H-piles shall be driven in the location shown on the plans. A rigid frame comprised of crossed timbers securely anchored, or other approved method of holding each pile in its proper location, shall be provided. Deviation of the location of the top of the pile from that shown on the plans shall not exceed two in. Piles shall be driven with a maximum permissible deviation of two in. in 10 ft from the vertical, or from the batter indicated on the plans. The hammer and pile shall be supported in rigid leads that extend to within two ft of the elevation at which the pile enters the ground. Before driving starts, the leads and pile shall be carefully plumbed. A suitable driving cap shall be provided to prevent undue damage to the top of the pile, and to hold the pile centered under the hammer.

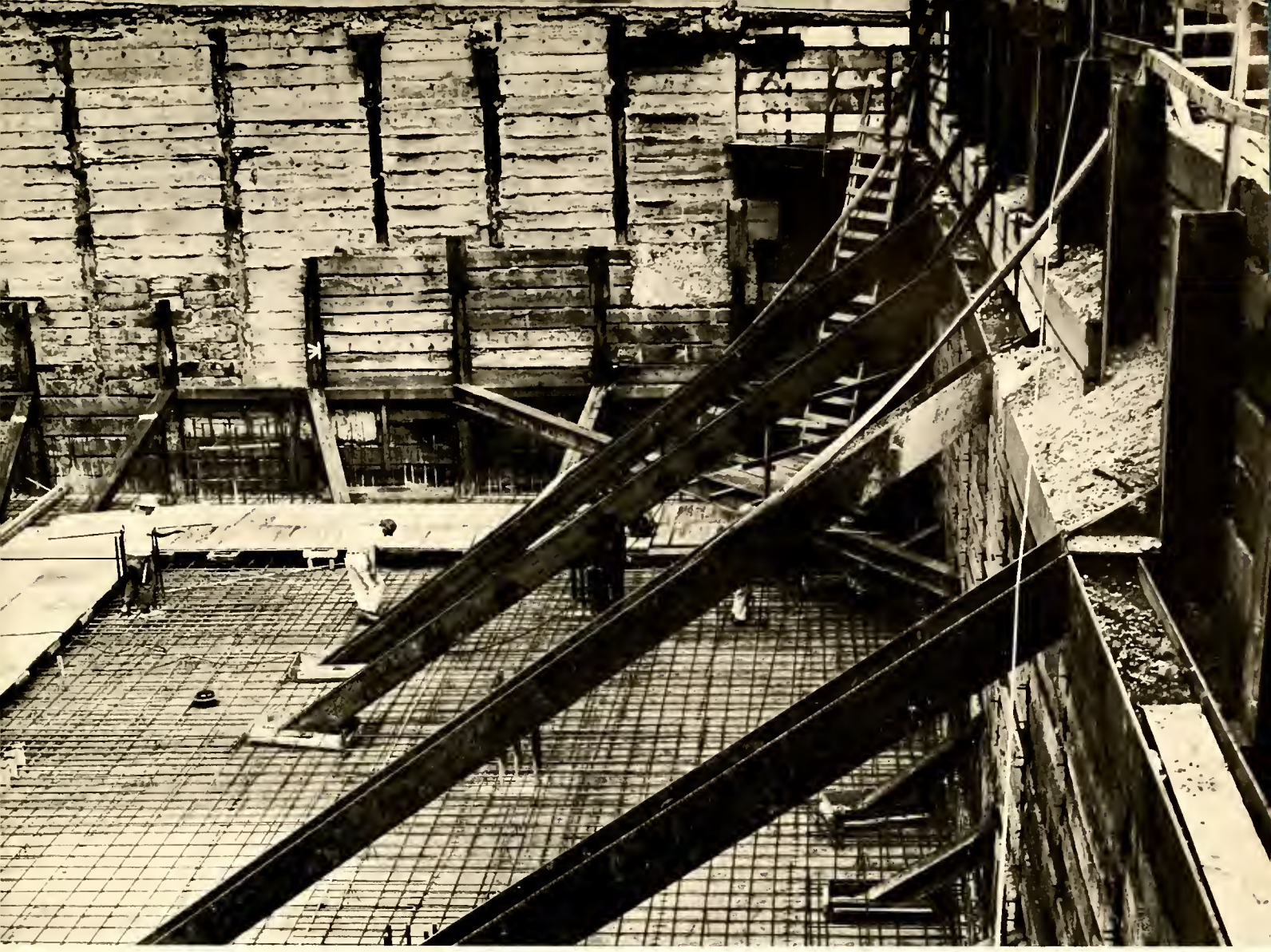
(*c*) *For cases where accurate location and plumbness of the pile are of particular importance, such as bridges or viaducts on H-pile bents or other cases where the piles form part of the superstructure.*

The H-piles shall be driven in the locations shown on the plans. A rigid frame shall be provided at ground level [*water level for a job in open water*] with a second corresponding frame about 20 ft higher. These frames shall be securely constructed so that each pile is held in its correct position. Any other adequate method of holding the piles in correct position may be accepted, subject to the approval of the Engineer. Deviation of the location of the top of the pile from that shown on the plans shall not exceed one in. Piles shall be driven with a maximum permissible deviation of a quarter-inch in 10 ft from the vertical, or from the batter indicated on the plans. The hammer shall be held squarely and securely on the top of the pile in a manner meeting the approval of the Engineer.

7. Specification for refusal on rock. A pile shall be considered driven to refusal when 5 blows of the hammer (with energy per blow equivalent to or exceeding that specified) produce a total penetration of a quarter-inch. Driving should then cease, provided that the pile has been driven to the depth at which the borings indicate rock. If the tip of the pile has not reached rock, an obstruction must be assumed and the method of disposition must be subject to approval of the Engineer.

8. Basis of payment. Steel H-piles shall be driven in lengths specified by the Engineer, and payment shall be based upon the lengths so driven and accepted.

Steel H-piles, including steel pile caps, points, and splices shall be paid for at the contract unit price per foot. Test load piles shall be paid for at the contract unit price for test load piles. These prices and payments shall constitute full compensation for furnishing all materials, labor, and the use of all tools, equipment, and incidentals necessary to complete this item.



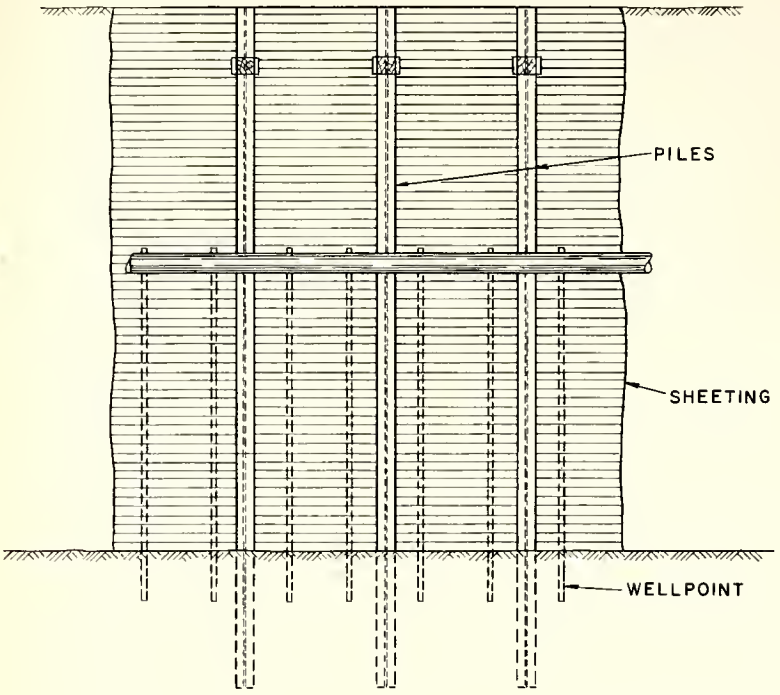
A soldier-beam excavation, with batter piles used as wales and diagonal bracing.

Section 17

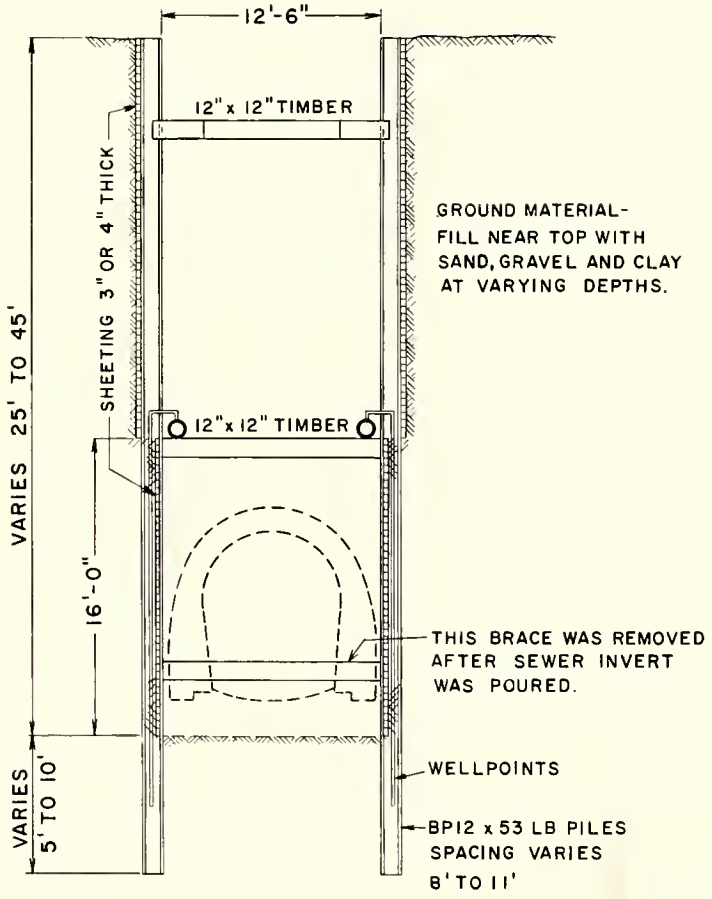
SOLDIER-BEAM METHOD OF EXCAVATION

An important use of H-piles is in open-cut excavating by the soldier-beam method, which has been widely used in constructing subways, tunnel approaches, large storm sewers, building foundations, and similar structures. It is applicable to open cuts in stiff clay, medium clay, sandy clay, or sand. Photographs and designs of the method are shown here.

Briefly, the method consists of driving vertical H-piles in line on each side



SIDE ELEVATION



TYPICAL SECTION

FIGURE 17A

of the cut on centers of six to 10 ft. Excavation is then started. As the H-piles are uncovered, the spaces between them are closed with horizontal timber lagging, the ends of which are placed behind and supported by the inner flanges of the H-piles. There are also patented attachments that permit the lagging to be applied to the face of the H-piles. In clay soils, it is usually possible to excavate the bay between any two H-piles for about five feet lower than the last placed lagging, and then to lag the face of the uncovered soil. In loose running sand, the lagging is placed as soon as there is space for it, to prevent the possibility of a cave-in.

In wide sand cuts, the excavation is carried lower in the center of the cut. This causes the ground water to drain out of berms left against the sides, and prevents the flow of water through the sand at the top of the berm where the lagging is being installed. In narrow trenches where this cannot be done, the water may be drained out in advance of the lagging operation with well points, as shown in Figure 17A.

Cross-bracing is placed horizontally between opposite H-piles. Where larger pockets are required, however, horizontal wales are placed against the H-piles, and the cross-struts are then placed horizontally between the wales at the desired intervals. In very large excavations, inclined shores may be used instead of cross-bracing.

Where the cut is so wide that the cross-braces require lateral support, it is usually furnished by longitudinal members between the braces. Vertical support for such bracing may be provided by temporary support piles driven before excavation is started. An alternative method is to hang the braces from beams that span the cut and rest on top of the H-pile soldier beams. A great advantage of the soldier-beam method in free-draining soils, is that the water is drained from the soil adjacent to the cut. This greatly reduces the pressure for which the bracing must be designed.

In general, the soldier-beam method is adaptable to a wide variety of conditions, with special methods of construction applicable to each job.

As the function of soldier beams is to resist lateral soil pressures, rather than the vertical loads of bearing piles, designs follow conventional methods for members in flexure. The points of support are the tiers of cross-bracing consisting of wales and struts; the spans between the supports are usually considered as simple beams.

Substantial economies often can be effected by

designing all soldier-beam components in one of Bethlehem's high-strength V Steels. The increased allowable unit stresses permitted by the use of these steels allow greater vertical spacing between bracing tiers, and greater spacing between cross-struts. This will, in turn, allow larger pockets for work areas. Alternatively, positioning of wales and struts can remain unchanged but members of lighter weight can be used.

A secondary but important advantage in using V Steel for soldier-beam work is the extremely high resistance to end damage under severe driving conditions. Very little, if any, material needs to be cut off the ends of V Steel piles due to driving damage, and they can be expected to last through many more re-uses than piles made of standard carbon steel. Minimum tensile strengths and yield points of Bethlehem V Steels are given in Table 17B.

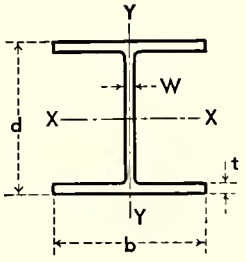
TABLE 17B

V Steel	*Section*	*Min tensile strength, psi*	*Yield point, psi*
V45	All	65,000	45,000
V50	All	70,000	50,000
V55	All	70,000	55,000
V60	All, except BP14x117	75,000	60,000

This excavation, entirely supported by Bethlehem H-piles used as soldier beams, measures 76 x 36 x 58 ft deep.

Section 18

QUICK FACTS ABOUT STEEL PILING AND HAMMERS



BETHLEHEM H-PILES

Theoretical Dimensions and Properties for Designing

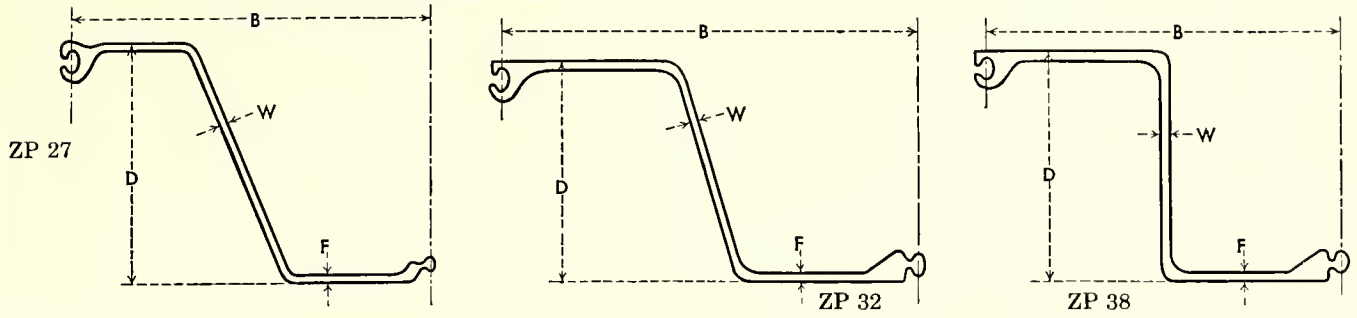
Section Number and Nominal Size	Wt. per Ft	Area of Section	Depth of Section	Flange		Web Thickness	Axis X-X			Axis Y-Y		
				Width	Thickness							
		A	d	b	t	W	I	S	r	I′	S′	r′
	lb	in.²	in.	in.	in.	in.	in.⁴	in.³	in.	in.⁴	in.³	in.
	117	34.44	14.234	14.885	0.805	0.805	1228.5	172.6	5.97	443.1	59.5	3.59
BP 14	102	30.01	14.032	14.784	0.704	0.704	1055.1	150.4	5.93	379.6	51.3	3.56
14 x 14½	89	26.19	13.856	14.696	0.616	0.616	909.1	131.2	5.89	326.2	44.4	3.53
	73	21.46	13.636	14.586	0.506	0.506	733.1	107.5	5.85	261.9	35.9	3.49
BP 12	74	21.76	12.122	12.217	0.607	0.607	566.5	93.5	5.10	184.7	30.2	2.91
12 x 12	53	15.58	11.780	12.046	0.436	0.436	394.8	67.0	5.03	127.3	21.2	2.86
BP 10	57	16.76	10.012	10.224	0.564	0.564	294.7	58.9	4.19	100.6	19.7	2.45
10 x 10	42	12.35	9.720	10.078	0.418	0.418	210.8	43.4	4.13	71.4	14.2	2.40
BP 8 8 x 8	36	10.60	8.026	8.158	0.446	0.446	119.8	29.9	3.36	40.4	9.9	1.95

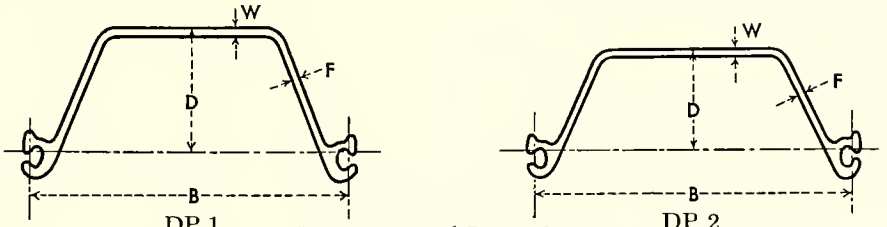
Available Grades

ASTM	Bethlehem V Steel			
A36	V45	V50	V55	V60
X	X	X	X	
X	X	X	X	X
X	X	X	X	X
X	X	X	X	X
X	X	X	X	X
X	X	X	X	X
X	X	X	X	X
X	X	X	X	X
X	X	X	X	X
X	X	X	X	X

BETHLEHEM STEEL SHEET PILING

Bethlehem Sheet piling is made in A328 steel, and in V45, V50, and V55 steels.

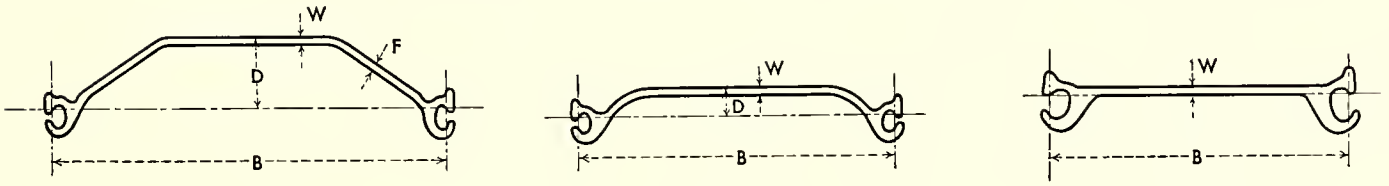




Dimensions and Properties

Section Number	Nominal Width B	Web Thickness W	Flange Thickness F	Nominal Depth D	Single Section: Weight per Foot	Single Section: Area	Single Section: I	Single Section: S	Per Foot of Wall: Weight, lb per sq ft	Per Foot of Wall: S
	in.	in.	in.	in.	lb	in.²	in.⁴	in.³		in.³
ZP 38	18	3/8	1/2	12	57.0	16.77	421.2	70.2	38.0	46.8
ZP 32	21	3/8	1/2	11 1/2	56.0	16.47	385.7	67.0	32.0	38.3
ZP 27	18	3/8	3/8	12	40.5	11.91	276.3	45.3	27.0	30.2
DP 1	16	31/64	3/8	6	42.7	12.56	87.0	20.4	32.0	15.3
DP 2	16	3/8	3/8	5	36.0	10.59	53.0	14.3	27.0	10.7

Note: I Moment of inertia about major axis X-X.
S Section modulus about major axis X-X.



Dimensions and Properties

Section Number	Nominal Width B	Web Thickness W	Flange Thickness F	Nominal Depth D	Single Section: Weight per Foot	Single Section: Area	Single Section: I	Single Section: S	Per Foot of Wall: Weight, lb per sq ft	Per Foot of Wall: S
	in.	in.	in.	in.	lb	in.²	in.⁴	in.³		in.³
AP 3	19 5/8	3/8	3/8	3 1/2	36.0	10.59	26.0	8.8	22.0	5.4
SP 4	16	3/8	..	1 11/32	30.7	8.99	5.5	3.2	23.0	2.4
SP 5	16	1/2	..	1 11/32	37.3	10.98	6.0	3.3	28.0	2.5
SP 6a	15	3/8	..	..	35.0	10.29	4.6	3.0	28.0	2.4
SP 7a	15	1/2	..	..	40.0	11.76	4.6	3.0	32.0	2.4

Note: I Moment of inertia about major axis X-X.
S Section modulus about major axis X-X.

PILE HAMMER SPECIFICATIONS

Rated Energy, ft lb	Mfr	Model	Type	Strokes per min	Stroke at Rated Energy, in.	Weight of Striking Parts, lb	Total Weight,[3] lb	Length at Hammer,[3] ft	Length at Hammer,[3] in.	Width at Hammer, in.	ASME Boiler hp	Nominal Boiler hp (12 sq ft)	Required Air, cfm	Steam or Air psi Required at Hammer, lb	Size of Hose, in.
130,000	M K T	S-40	Sgl-Act Stm-Air	55	39	40,000	96,000	16	0	60	375	—	—	150	4
113,488	Vulcan	400C	Differential Stm-Air	100	16½	40,000	83,000	16	8¾	50	700	300	4,659	150	5
97,500	M K T	S-30	Sgl-Act Stm-Air	60	39	30,000	86,000	16	0	60	280	—	—	113	4
60,000[1]	M K T	S-20	Sgl-Act Stm-Air	60	36	20,000	38,650	15	5	36	190	150	—	150	3
60,000	Vulcan	020	Sgl-Act Stm-Air	60	36	20,000	39,000	15	0	37	217	105	1,634	120	3
50,200	Vulcan	200C	Differential Stm-Air	98	15½	20,000	39,050	13	2	37	260	120	1,746	142	4
48,750	Vulcan	016	Sgl-Act Stm-Air	60	36	16,250	30,250	14	6	32	210	105	1,290	120	3
42,000	Vulcan	014	Sgl-Act Stm-Air	60	36	14,000	27,500	14	6	32	200	100	1,282	110	3
39,800	Delmag	D-22	Sgl-Act Diesel	42-60	n/a	4,850	10,054	12	10½	24	—	—	—	—	—
37,500[1]	M K T	S-14	Sgl-Act Stm-Air	60	32	14,000	31,700	13	7	36	155	90	—	100	3
36,000	Vulcan	140C	Differential Stm-Air	103	15½	14,000	27,984	12	3	32	211	100	1,425	140	3
32,500[1]	M K T	S-10	Sgl-Act Stm-Air	55	39	10,000	22,380	13	0	30	130	65	1,000	80	2½
32,500	Vulcan	010	Sgl-Act Stm-Air	50	39	10,000	18,750	15	0	26	157	75	1,002	105	2½
32,000	M K T	DE-40	Sgl-Act Diesel	48 avg	96	4,000	11,275	15	0	26	—	—	—	—	—
30,225	Vulcan	OR	Sgl-Act Stm-Air	50	39	9,300	16,765	15	0	26	—	72	1,020	100	2½
26,300	Link Belt[4]	520	Sgl-Act Diesel	80-84	43.17	5,070	12,545	13	6	26	—	—	—	—	—
26,000[1]	M K T	S-8	Sgl-Act Stm-Air	55	39	8,000	18,300	13	3	26	120	60	850	80	2½
26,000[1]	M K T	C-8	Dbl-Act Stm-Air	77-85	20	8,000	18,750	9	9	30	110	60	875	100	2½
26,000	Vulcan	08	Sgl-Act Stm-Air	50	39	8,000	16,750	15	0	26	127	60	880	83	2½
24,450	Vulcan	80C	Differential Stm-Air	111	16½	8,000	17,885	12	1⅝	26	180	80	1,245	120	2½
24,450	Vulcan	8M	Differential Stm-Air	111	n/a	8,000	18,400	10	6	32	180	80	1,245	120	2½
24,370	Vulcan	O	Sgl-Act Stm-Air	50	39	7,500	16,250	15	0	26	—	60	841	80	2½
24,000	M K T	C-826	Dbl-Act Stm-Air	85-95	18	8,000	17,750	12	2	26	120	—	875	125	2½
22,600	Delmag	D-12	Sgl-Act Diesel	42-60	n/a	2,750	5,440	12	7⅜	—	—	—	—	—	—
22,400	M K T	DE-30	Sgl-Act Diesel	48 avg	96	2,800	9,075	15	0	20	—	—	—	—	—
19,500	Vulcan	06	Sgl-Act Stm-Air	60	36	6,500	11,200	13	0	20	94	50	625	100	2
19,200	Vulcan	65C	Differential Stm-Air	117	15½	6,500	14,886	12	1½	20	152	70	991	150	2
19,150[1]	M K T	11B3	Dbl-Act Stm-Air	95	19	5,000	14,000	11	1½	26	126	60	900	100	2½
16,250	M K T	S-5	Sgl-Act Stm-Air	60	39	5,000	12,460	12	2	24	85	40	600	80	2
16,000	M K T	DE-20	Sgl-Act Diesel	48 avg	96	2,000	6,325	13	3	20	—	—	—	—	—
16,000[1]	M K T	C-5	Dbl-Act Stm-Air	100-110	18	5,000	11,880	8	9	26	80	45	585	100	2½
15,100	Vulcan	50C	Differential Stm-Air	120	15½	5,000	11,782	11	⅜	20	125	60	880	120	2
15,100	Vulcan	5M	Differential Stm-Air	120	n/a	5,000	12,900	9	4	28	125	60	880	120	2
15,000	Vulcan	1	Sgl-Act Stm-Air	60	36	5,000	9,700	13	0	20	81	40	565	80	2
15,000	Link Belt[4]	312	Sgl-Act Diesel	100-105	30.89	3,857	10.375	10	9	26	—	—	—	—	—
13,100[1]	M K T	10B3	Dbl-Act Stm-Air	105	19	3,000	10,850	9	2	24	104	50	750	100	2½
9,040	Delmag	D5	Sgl-Act Diesel	42-60	n/a	1,100	2,401	11	2½	—	—	—	—	—	—
9,000[1]	M K T	C-3	Dbl-Act Stm-Air	130-140	16	3,000	8,500	7	9½	20	60	30	450	100	2
9,000[1]	M K T	S-3	Sgl-Act Stm-Air	65	36	3,000	9,030	11	4	20	57	25	400	80	1½
8,800	M K T	DE-10	Sgl-Act Diesel	48 avg	96	1,100	3,518	12	2	14½	—	—	—	—	—
8,750[1]	M K T	9B3	Dbl-Act Stm-Air	145	17	1,600	7,000	8	3¾	20	85	45	600	100	2
8,100	Link Belt[4]	180	Sgl-Act Diesel	90-95	37.60	1,725	4,550	11	3	18	—	—	—	—	—
7,260	Vulcan	30C	Differential Stm-Air	133	12½	3,000	7,036	9	8⅛	19	70	40	488	120	1½
7,260	Vulcan	2	Sgl-Act Stm-Air	70	29¾	3,000	6,700	11	6	19	49	25	336	80	1½
7,260	Vulcan	3M	Differential Stm-Air	133	n/a	3,000	8,490	7	11	24	70	40	488	120	1½
6,500	Link Belt[4]	105	Sgl-Act Diesel	90-98	35.23	1,445	3,885	10	3	18	—	—	—	—	—
4,150[2]	M K T	7	Dbl-Act Stm-Air	225	9½	800	5,000	6	½	21	65	35	450	100	1½
4,000	Vulcan	DGH-900	Differential Stm-Air	238	10	900	5,000	6	9	18	75	40	580	78	1½

NOTES: 1. Hammer suitable for underwater driving.

2. Hammer may be converted to an extractor.

3. McKiernan-Terry steam-air hammers include flat anvil. Vulcan hammers include standard base but no helmet. Diesels shown do not include drive caps.

4. Link-Belt energy ratings are based on equivalent WH. Output gages are available to determine energy ratings under any driving conditions.

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